Shear Strength of RC Beam Made with Recycled Brick Aggregate

by

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Islamic University of Technology

Organization of Islamic Cooperation

MASTER OF SCIENCE IN CIVIL ENGINEERING

April, 2019

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A Thesis Submitted to the

Department of Civil and Environmental Engineering

Islamic University of Technology

Organization of Islamic Cooperation

In Partial Fulfillment for

The degree of Master of Science in Civil Engineering

Recommendation of the Board of Examiners

The thesis titled "**Shear Strength of RC Beam Made with Recycled Brick Aggregate**" submitted by Md. Kamal Hossain Shikdar, Student ID 155607 of Academic Year 2015-2016 has been found as satisfactory and accepted as partial fulfillment of the requirement for the degree of Master of Science in Civil Engineering.

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Dedication

I would like to dedicate this thesis to my parents, my grandparents and all my teachers who brought me up to this moment.

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Symbol	Description
A	Unit content of coarse aggregate
A_{st}	*A*rea of the tensile reinforcement,
$Air\ (\%)$	Percentage of air in concrete
ACI	American Concrete Institute
a_s	Shear span
b	width of the beam
BA	Brick Aggregate
BNBC	Bangladesh National Building Code
b_v	Effective width of the
b_w	Width of the member
C	Unit content of cement
d	Effective depth
d_a	Maximum aggregate size,
d_v	Effective shear depth taken as the greater of 0.9d or 0.72h
d_{agg}	Maximum aggregate size of concrete
E_s	Modulus of elasticity of Steel
E_s	Modulus of elasticity of reinforcing bars
f'_c	Compressive strength of concrete
f_{ck}	Characteristic value of compressive strength of concrete,
f_{ck}	Compressive strength of concrete
G_A	Specific gravity of coarse aggregate (SSD)
G_c	Specific gravity of cement

G_s	Specific gravity of fine aggregate (SSD)
G_w	Specific gravity of water
M	Bending moment
NA	Natural aggregate
RBA	Recycled Brick aggregate
RC	Reinforced Concrete
RCA	Recycled Concrete Aggregate
S	Unit content of fine aggregate
$V_{Rk.c}$	Shear capacity.
V_c	Shear capacity of concrete
V	Total shear force
V_{cr}	Critical shear strength
$V_{Rd,c}$	Shear resistance
W	Unit content of water
z	Effective shear depth, the partial safety factor,
ε_s	Measured longitudinal strain at the bottom steel reinforcement
β	Factor indicating the ability of diagonal cracked concrete to transmit tension,
γ_w	Unit weight of water
γ_w	A safety factor
γ_c	Partial safety factor
ρ	longitudinal reinforcement ratio

Acknowledgments

Glory and praise be to Allah the Almighty for giving me the strength and courage to successfully complete my M.Sc. thesis. Verily there is no strength except with Allah.

First of all, I would like to acknowledge the support and motivation of my thesis supervisor Prof. Dr. Md. Tarek Uddin, to whom, I owe my sincere appreciation for letting me be a part of his research project upon which this thesis is based. He has been a true inspiration that led me into the research field of Concrete Technology. It is only through his concern, encouragement, and motivation that this work was completed. I am forever indebted.

A note of thanks is also extended to Prof. Dr. Md. Rezaul Karim, Prof. Dr. Md. Nazrul Islam and Prof. Dr. Hossain Md. Shahin, who served on my Master's Thesis Committee, for their time and thoughtful review.

I wish to express my sincerest thanks and appreciation to my research group members, laboratory technicians, and laboratory assistants of the Concrete Lab of the Department of CEE, IUT for their constant help.

Finally, I am forever indebted to my parents, who raised me with love and supported me in all my pursuits, and for their understanding, endless patience, and encouragement. They form part of my vision that defined the good things that really matter in life.

I also acknowledge the financial support provided by Structural Engineers Limited (SEL) to conduct this study. And also acknowledge the laboratory and other associated facilities provided by the Department of Civil and Environmental Engineering, Islamic University of Technology (IUT), Gazipur to carry out this study.

ABSTRACT

In the last couple of decades, however, there has emerged greater importance on sustainability. Unfortunately, concrete, our most common construction material uses a significant amount of non-renewable resources. Consequently, recycled brick aggregate (RBA) has been considered as an alternative construction material to conventional natural aggregates. Most research to date has consisted only of the evaluation of the strength and durability of RBA mixtures, while only a limited number of studies have implemented full-scale testing of specimens constructed with RBA to determine its potential use in the industry.

For this research, the shear strength of reinforced concrete (RC) beams made with recycled brick aggregate was investigated. As control specimens, RC beams made with virgin brick aggregate were also investigated. For the investigation, 32 reinforced concrete beams of size 200 mm by 300 mm by 2100 mm and 200 mm by 300 mm by 2400 mm were made. The variables were steel ratio (0.82% and 1.23%), shear span to depth ratio (2.04 and 2.45), and compressive strength of concrete (24 MPa and 29 MPa). In the shear span of the beam specimens, shear reinforcements were not provided and the beams were designed to ensure shear failure according to ACI 318-14. Shear strength of the beams without shear reinforcement was evaluated by four-point loading test. Shear strength of concrete beams was also evaluated by using different codes and fracture mechanics approaches. These results were compared with the experimental results. The results obtained from this study were also compared with the shear database.

It is revealed that the RC beam made with recycled brick aggregate shows similar shear strength as of RC beam made with virgin brick aggregate. Existing code provisions, as well as fracture mechanics approach, can be used for predicting shear strength of concrete beams made with recycled brick aggregate.

CHAPTER 1: INTRODUCTION

1.1 GENERAL

Concrete is universally used construction material in all types of structures. According to Neville (Neville, 1995), it typically contains about 12 % cement and 80% aggregate by mass. In the fresh state, the global construction industry uses approximately 1.6 billion tons of cement and 10 billion tons of sand, gravel, and crushed rock annually (Mehta, 2001). The extraction of natural aggregates has several negative environmental consequences such as the destruction of the natural habitat and loss of the water storage capacity of the ground (Winfield & Taylor, 2005). Therefore, by taking into consideration the needs of future generations, the construction industry has to play a significant role in sustainable development. With these growing environmental awareness, the world is increasingly turning to research recycled materials.

In this chapter, background information on the subject is provided, as well as the problem statement, objectives and scope of the study. Organization of the thesis into chapters is also included.

1.2 BACKGROUND

Bangladesh is predominantly flat delta land. It has very limited resources of stone aggregate for making concrete. Due to the economic growth as well as increase in population, the demand of volume of construction materials is increasing every year. On the other hand, in many cases, existing buildings were demolished due to deterioration by carbonation or chloride induced corrosion of steel in concrete (Mohammed, et al., 2007).

To understand the possibility of utilization of demolished concrete made with brick aggregate as coarse aggregate for new construction works, an extensive study was conducted (Mohammed, et al., 2015). Demolished concrete blocks were collected from 33 different demolished building sites and investigated for different properties. Also, concrete specimens were made and tested for different mechanical properties. It was revealed that demolished concrete made with recycled brick aggregate can be utilized for making structural concrete of strength 20 MPa to 30 MPa.

Further investigations were also conducted to understand the flexural behavior of RC concrete beams made with recycled brick aggregate and virgin brick aggregate (Mohammed, et al., 2017). No significant difference of test results was found between RC beams made with recycled brick aggregate and virgin brick aggregate. Also, it was found that guideline of existing ACI code can be used for the design of RC beams made with RBA.

However, currently, in Bangladesh, there are no established guidelines for producing structural concrete by using recycled concrete aggregate (RCA). For the consistent production, quality control, and design, its use as structural material is lacking. With this view, a research project was undertaken, to study the structural property (shear strength) of RC beam made with recycled brick aggregate (RBA).

1.3 RESEARCH SIGNIFICANCE

There have been many published studies in the global literature on the mix design, mechanical properties, and durability of concrete made with recycled materials (Sheen, et al., 2013; Radonjanin, et al., 2013; Corinaldesi & Moriconi, 2009; Etxeberria, et al., 2007; Corinaldesi, 2010). Work on structural behavior, structural member strength characteristics, and structural design recommendations lacks far behind research work at the material level. The flexural strength of reinforced concrete beams does not depend much on the mechanical properties of the concrete. The axial capacity of reinforced concrete tied or spiral columns can be somewhat predicted from the steel and concrete material strengths. However, shear strength of concrete inside structural elements is a very complex phenomenon that often cannot be extrapolated from the properties of the involved materials with ease. Based on the above, there is a need to investigate the shear strength of reinforced concrete members made with recycled brick aggregate. In Bangladesh, The existing design codes (particularly Bangladesh National Building Code (BNBC, 2015) and (ACI318, 2014)) which are developed based on the research results on stone aggregate are generally used for strength evaluation as well as the design of RC members made with brick aggregate (BA). However, it is necessary to validate the equations of different codes for shear capacity of RC beams made with BA as well as RBA. It is also necessary to compare the experimental results of the shear capacity of RC beams with the estimated shear capacity obtained from equations formulated based on fracture mechanics approach by different researchers. Moreover, it is also necessary to compare the diagonal shear capacity of the RC beams with the existing results of the shear database with the variation of shear-span-to-depth ratio, effective depth, steel ratio, and compressive strength of concrete.

1.4 OBJECTIVES OF THE STUDY

The objective of this study is to use of recycled brick aggregate as replacement of natural aggregate for saving the natural resources and energy. The aim for this work is to investigate the shear behavior of RBA rectangular beams without stirrups with shear span to effective depth (a/d) ratio , to experimentally investigate the variation of shear strength due to variation in main tensile reinforcement, to compare the shear strength with the compressive strength of concrete and to compare the shear strength of RBA with the available code provisions for normal concrete.

The main objective of this research study was to evaluate the shear behavior and response of RBA through material, component, and full-scale testing.

The objectives of this study are as follows:

(i) To investigate the shear capacity of the RC beams made with RBA
(ii) To compare the shear strength of RBA with BA
(iii) To compare the shear capacity of RBA with provisions of existing codes and equations developed from the fracture mechanics approaches
(iv) To compare the results of this study with the shear database and understand the crack morphology, crack progression of the RBA.

1.5 SCOPE OF WORK

The following scope of work was implemented in order to achieve the objective of the research study:

- Perform a literature review
- Develop a research plan
- Develop mix designs for both BA and RBA
- Evaluate the fresh and hardened properties of BA and RBA
- Design and construct small and full-scale specimens
- Test specimens to failure
- Record and analyze data from tests
- Compare test results to current guidelines and previous research findings
- Develop conclusions and recommendations
- Prepare this report to document the details, results, findings, conclusions, and recommendations of this study.

1.6 RESEARCH METHODOLOGY

The proposed research methodology (a flow chart is shown in **Figure 1.1**) included seven tasks necessary to successfully complete the study. They are as follows:

1. Perform a literature review

   The goal of the literature review was to become familiarized with testing methods and results from previous studies. This knowledge was used for a better understanding of the behavior of the specimens, to avoid mistakes, as well as to provide support for comparisons.

2. Collection of Materials

   Recycling aggregate was collected from a demolished site (**Figure 1.2**) situated at Gulshan circle -1, Dhaka, Bangladesh. A thirty years old building was demolishing for building 20 storied commercial building instead of present 10 storied building. After collecting the concrete block, recycled aggregate concrete (RAC) was separated and sized into desired size (20mm down). Cement was supplied by Seven Rings cement (A Cement Company). The deformed bar was bought from BSRM steels. Sand and bricks were collected from the local market.

3. Perform material and component testing

   A number of hardened concrete property tests were completed to evaluate the performance of the RBA mix and determine the validity of using these tests to predict the performance of concretes containing a recycled concrete aggregate.

4. Mix design

   Develop RBA and BA mix designs. Both mix designs served as controls during this study.

5. Carry out full-scale testing

   This task was critical as current shear design provisions for reinforced concrete are largely empirical. This task involved the construction and testing of full-scale specimens to confirm the potential of RBA. The full-scale specimens included beam specimens for shear testing only. In order to compare the shear strength of conventional and RBA, full-scale beams were tested in a two point loading configuration. These beams were designed to fail in shear. Different longitudinal reinforcement ratios were also considered. Strain gauges were applied to the flexural reinforcement, and the maximum load applied to the beam was also recorded and used to calculate the shear strength of the beams.

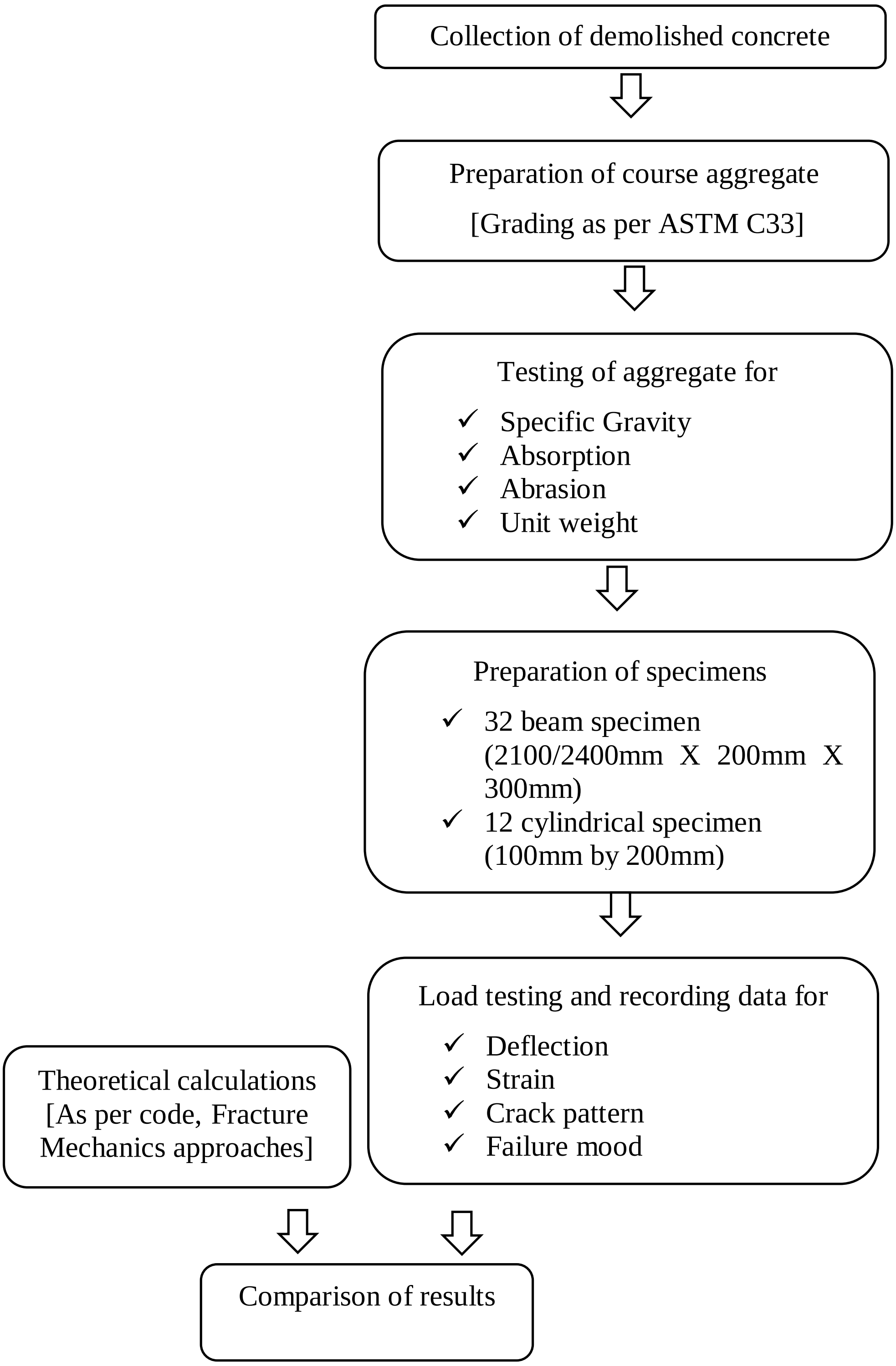


Figure 1.1: Flow chart of research methodology

Figure 1.2: Demolished site

6. Analyze test data

   The material, component, and full-scale test results were analyzed to evaluate the shear behavior and response of RBA compared to BA. The test data included: concrete compressive and tensile strength, shear force-deflection plots, crack formation and propagation, and reinforcement strains.

7. Develop findings conclusions, and recommendations.

   This task synthesized the results of the previous tasks into findings, conclusions, and recommendations on the shear behavior and response of RBA.

1.7 REPORT OUTLINE

This report includes five chapters. This section will discuss the information that will be presented in more detail throughout this document.

Chapter 1 demonstrates as an introduction to the report. This introduction contains a brief background of RBA. It also discusses the research objective, scope of work, and research plan.

Chapter 2 includes information from previous research performed on the characterization of brick (both recycled and fresh) aggregate and its applications as coarse aggregate in concrete.

Chapter 3 includes information about the experimental program. The experimental program consisted of 32 tests performed on full-scale reinforced concrete beams as well as material and component testing to determine hardened concrete properties such as compressive strength, splitting tensile strength and flexural strength. This chapter also describes the fabrication process, test set-up, and instrumentation for full-scale testing.

Chapter 4 presents the test results and the different analyses used to investigate the shear resistance Capacity. The overall behavior of the specimens is described first, with a focus on crack patterns, failure modes, and shear strength.

Chapter 5 concludes this document, summarizing the findings and conclusions of this study and proposing recommendations for future research.

CHAPTER 2: LITERATURE REVIEW

2.1 GENERAL

The global demand for regulating, recycling and reusing construction and demolition waste has increased in the last few decades due to both economic and environmental considerations and represents the facts that are responsible for the search of alternative materials. Therefore, this chapter provides a review of previous work covering materials property and shear capacity of stone, fresh brick and recycled brick aggregate, as the coarse aggregate in new concrete and the prospects of using such a material in modern construction.

2.2 USE OF RECYCLED AGGREGATE AS COARSE AGGREGATE

Recently, there has been an increasing trend toward the use of sustainable materials. Sustainability helps the environment by reducing the consumption of non-renewable natural resources. Concrete – the second most consumed material in the world after water – uses a significant amount of non-renewable resources. As a result, numerous researchers have investigated the use of recycled materials in the production of concrete such as fly ash and recycled aggregate.

Unfortunately, global data on concrete waste generation is not available, but construction and demolition waste account for around 900 million tones every year just in Europe, the US, and Japan (WBCSD, 2009). Recycling concrete not only reduces using virgin aggregate but also decreases the amount of waste in landfills.

Recycled Brick Aggregate (RBA) is composed of both the original aggregate and mortar that remains stick to the surface of the aggregate. In the production of RBA, the removal of all this residual mortar would prove costly and detrimental to the integrity of the virgin aggregates within the concrete. Therefore, compared to natural aggregates residual mortar is unavoidable. Research has shown that this residual mortar causes high water absorption, low density, low specific gravity, and high porosity. These effects in the recycled aggregate can decrease hardened concrete properties of recycled aggregate concrete (RAC).

2.3 MATERIAL PROPERTY OF FRESH BRICK AGGREGATE

Since early Roman times, concrete buildings made with crushed brick have been known. An early example of this is the concrete channels of the Eiffel water supply to Cologne. In this structure, the binder is a mixture of lime and crushed brick dust or other pozzolans of the time (Czernin, 1980; Hansen, 1992)

Hansen (1992) found that the first recorded mixing of crushed brick concrete with Portland cement was in Germany from 1860 for the manufacture of concrete products. Systematic investigations have been carried out since 1928 on the effect of the cement content, water content and grading of crushed brick. Although, the first significant applications of crushed brick aggregate only date back to the use of rubble from buildings destroyed in the Second World War (Hansen, 1992).

Broken brick along with burnt clay and earthenware were also used in the UK from the late 1800s, but even then their use was limited as engineers at the time recognized that the material did not have a high density (Cunningham, 1918).

The properties of brick aggregate concrete are as follows

2.3.1 Elasticity and Drying Shrinkage

The modulus of elasticity of crushed brick concrete is only between half and two thirds that of normal concrete of the same strength (Hansen, 1992). This can be compared with values reported by Hansen & Boegh, (1985) who produced concrete with crushed concrete as the coarse aggregate. Their tests showed that the modulus of elasticity for concrete containing crushed concrete as the coarse aggregate is up to 30% lower than that of normal concrete. This figure can even be lower according to Frondistou-Yannas, (1977) who reported a 40% reduction in the modulus of elasticity for recycled aggregate concrete with crushed concrete as the coarse aggregate. Also, results from these authors show that drying shrinkage and creep in concrete containing either concrete or masonry recycled aggregates is increased. These results show that the properties of crushed concrete and crushed brick aggregate concrete are similar although it is generally accepted that clean crushed concrete performs slightly better than crushed masonry when used as the coarse aggregate in concrete.

2.3.2 Water Penetration and Absorption

The surface skin of concrete is the first line of defense against the ingress of aggressive agents. The main agencies of deterioration of concretes require the presence and movement of water within the material itself. The measurement of well-defined material properties which describe the ability of concrete to absorb and transmit water by capillarity is an important part of assessing the probable durability of a concrete (Wilson, et al., 1998). Water is a necessary ingredient for the corrosion of embedded steel as it can carry chlorides and sulphates as well as other harmful ions. The presence of water can also cause freeze-thaw damage to concrete (Price & Bamforth, 1993). It is a material property which can be measured easily on its own by measurement of the capillary rise absorption rate.

Previous tests (Hansen, 1992) have shown that water penetration depths are 50% higher in crushed brick aggregate concretes than normal aggregate concretes. This is an important factor because the penetration of the concrete cover by water containing chlorides can result in corrosion of the reinforcing bars (McCarter, et al., 1992). Therefore, the cover to the reinforcement should be increased when using crushed brick aggregate concrete.

2.3.3 Density

The relative densities of the crushed brick aggregates and the recycled aggregates are considerably less than the density of the granite aggregate (Venny, 1999). This means that when these aggregates are used to produce concrete, the density of the concrete will be much lower as the aggregate density has a large influence on the concrete density.

2.3.4 Tensile Splitting Strengths

It was found, that in general, concretes containing recycled aggregates had lower tensile splitting strengths than concretes produced with granite or new brick aggregates. The recycled washed aggregate had a tensile splitting strength which was almost 40% less than the granite aggregate concrete and 25% less than the 10 holes new brick aggregate concrete. The concrete containing the recycled masonry aggregate performed better with a tensile splitting strength which was only 9% less than the 10 whole new brick aggregate concrete. This suggests that the impurities in the recycled washed aggregate cause significant losses in tensile splitting strength By (Venny, 1999).

Khaloo (1994) found that there is an increase in tensile strength of around 2% in crushed brick aggregate concrete compared with concrete made with natural aggregates. They put this down to the rough surface of the crushed brick which provides a better bond between the concrete matrix and the coarse aggregate.

2.3.5 Compressive strength of brick aggregate concrete

Zakaria and Carbrera (1996) investigated concrete containing crushed brick as the coarse aggregate. Their findings were that crushed brick aggregate concrete had a relatively lower strength at early ages than normal aggregate concrete. The authors attributed this characteristic to the higher water absorption of crushed brick aggregate compared with gravel which was used as the control aggregate. However, their investigation also found that crushed brick aggregate concrete had a relatively higher strength at later ages which they attributed to the pozzolanic effect of the finely ground portion of the brick aggregate.

Khaloo used crushed clinker bricks as the coarse aggregate in concrete. He reported only a 7% loss in concrete compressive strength compared with concrete made with natural aggregates. In contrast to this decrease in strength, there is a decrease in the unit weight of crushed brick concrete of 9.5%.

More recently, Akhtaruzzaman and Hasnat carried out some research using well-burnt brick as coarse aggregate in concrete. They found that it was possible to achieve concrete of high strength using crushed brick as the coarse aggregate. Their research was mainly focused on determining the mechanical properties of brick aggregate concrete, rather than the properties of the brick aggregate itself.

2.3.6 Structural Behavior (Flexural strength)

The theoretical flexural strength was calculated using standard equations for normal weight concrete beams. When compared to the experimental results, the values obtained for brick aggregate concrete beams were in close proximity to the computed theoretical values. This means that when designing a brick aggregate concrete beam for flexural strength, the standard equations for normal weight concrete should be used as the relationship is the same.

Khaloo (1994) reported a 15% increase in flexural strength which they also think is due to the improved bond between the cement paste and the coarse aggregate

This can be compared with Hansen (1992) who reported a 10% increase in tensile and flexural strengths when using crushed brick as the aggregate in concrete compared with normal aggregates. He also reported that flexural strength increased linearly as compressive strength increased when using crushed brick aggregate to produce concrete. This relationship for crushed brick aggregate concrete can be seen in **Figure 2.1**

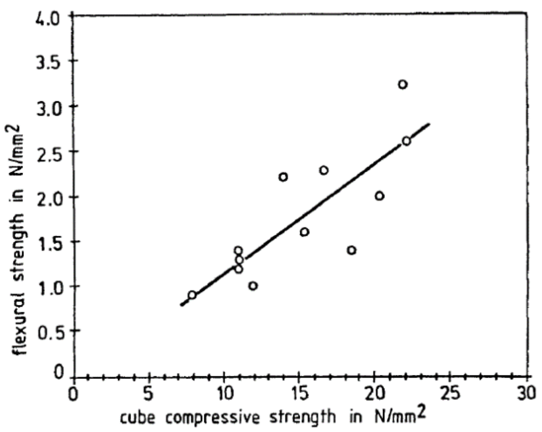


Figure 2.1: Relationship between flexural and compressive strength for crushed brick aggregate concrete

2.4 BASIC PROPERTIES OF CONCRETE MADE WITH RECYCLED CONCRETE AGGREGATE

In order to evaluate the structural behavior of a member, first one should have a good knowledge of the compressive and tensile strengths, elastic modulus, creep, and shrinkage characteristics of the materials from which it will be made.

When demolished concrete is crushed, a certain amount of mortar and cement paste from the original concrete remains attached to recycled aggregate. This attached mortar is the main reason for the lower quality of RCA compared to natural aggregate (NA). RCA compared to NA has the following properties:

Normal properties of recycled coarse aggregates are a lower density, a lower elastic modulus and a higher absorption than natural coarse aggregates. These properties are due to the fact that the recycled coarse aggregates have partially adhered mortar which has a relatively high volume of porosity (Roumiana, et al., 2003; Torben, 1986)

Researchers (Hansen, 1992; Sanchez & Gutierrez, 2004) found the lower bulk density of RCA compares to natural concrete. Lower specific gravity also recorded by researcher (Hansen, 1992). On the other hand, higher abrasion value were came up by the investigators (Hansen, 1992; Lopez-Gayarre, et al., 2009; Poon, et al., 2003) .

The recycled aggregates have less crushing strength, impact resistance, and specific gravity and have more absorption value as compared to fresh aggregates. Other properties are as follows.

- Increased crushability (Hansen, 1992),
- Increased quantity of dust particles (Hansen, 1992),
- Increased quantity of organic impurities if concrete is mixed with earth during building demolition (Hansen, 1992), and
- Possible content of chemically harmful substances, depending on service conditions in the building from which the demolition and crushing recycled aggregate is obtained (Hansen, 1992).

2.4.1 Compressive strength

The compressive strength of RAC using only coarse RCA has been found to be generally lower compared to the strength of similar control mixes of conventional concrete (Nixon, 1977; Hansen, 1986; Hansen, 1992; De Vries, 1993; Topcu & Guncan, 1995; Kiuchi & Horiuchi, 2003; Maruyama, et al., 2004; Topcu & Sengel, 2004). On the other hand, identical or even higher compressive strength in RAC, compared to a similar concrete made with NA (NAC), has been achieved by adjusting the w/c ratio (either by water content, cement content, or water and cement content ratio) as a function of the RCA content (Limbachiya, et al., 2000; Dhir, et al., 1999).

Gumede and Franklin collected recycled coarse aggregates (RCA) from a landfill site and used in lieu of natural coarse aggregates for the manufacture of concrete. With reference to the natural coarse aggregate, RCA replacement levels of 0%, 20%, 40%, 60%, 80%, and 100% were utilized and the results of hardened concrete tests for compressive and flexural strengths were obtained. It was found that in general the compressive and flexural strengths of the recycled aggregate concrete (RAC) decreased with increasing replacement levels of natural coarse aggregates using RCA. It was concluded that RCA could be employed as a substitute for natural aggregate in concrete only up to a certain limit or partial replacement. In this respect, it was also noted that the undesirable properties of RCA were primarily due to the quantity and quality of the adhering mortar.

2.4.2 Splitting tensile strength

According to Hansen, (1992), the relationship between c/w and compressive and splitting tensile strengths for RAC is linear as for normal-strength (30-70MPa) concrete with decreased splitting and flexural tensile strength up to 10% .

2.4.3 Modulus of elasticity

It has been shown that due to a large amount of attached residual mortar, the elastic modulus of RAC is always lower than that of companion NAC, while its drying shrinkage and creep are always higher than those of NAC (Hansen, 1986; Yamasaki & Tatematsu, 1998; Limbachiya, et al., 2000; Kiuchi, 2001; Kiuchi & Horiuchi, 2003; Gomez-Soberon, 2003; Maruyama, et al., 2004; Sakata & Ayano, 2000). Rahal, 2007 and Yang et al., 2008 exactly calculated the reduction of the modulus of elasticity which is up to 45%,

2.4.4 Water absorption

The technology of RAC production is different from the production procedure for concrete with natural aggregate. Because of the attached mortar, recycled aggregate has significantly higher water absorption than the natural aggregate. Yang et al. investigated for compressive and tensile strengths, moduli of rupture and elasticity, and shrinkage strain. They tested the properties of fresh and hardened concrete together with a comprehensive database according to the relative water absorption of aggregates combining the quality and volume of recycled aggregates used. In addition, the properties of hardened concrete with different replacement level and quality of recycled aggregates were compared with design equations proposed by ACI 318-05 and others. Their test results clearly showed that the properties of fresh and hardened concrete containing recycled aggregates were significantly dependent on the relative water absorption of aggregates. In addition, the moduli of rupture and elasticity of recycled aggregate concrete were lower than the design equations specified in ACI 318-05, when the relative water absorption of aggregates is above 2.5 and 3.0, respectively

Due to the high water absorption of RCA, Fumoto and Yamada (2003) suggested that the quantity of water absorbed by RCA be added to the total water required in concrete. Water absorption increased up to 50% (Hansen et al. 1992). Water absorption capacity higher in RCA than fresh concrete.

2.4.5 Creep and shrinkage of recycled aggregate concrete

Creep is the increase in strain under sustained stress, which in the case of concrete can be several times as large as the initial strain. It should be noted that it is the hydrated cement paste which undergoes creep, while the aggregate in concrete restrains creep. Thus, creep is a function of the cement paste volume in concrete, but there are certain physical properties of aggregate which also influence the creep of concrete (Neville, 1995). Aggregates elastic modulus is an important factor because stiffer aggregates restrain creep more effectively.

Concrete will contract on drying irrespective of the applied stress, and the magnitude of shrinkage is of the same order as the elastic strain under the usual range of stress (Neville, 1995).

However, partial replacement of 20-30% coarse NA by coarse RCA does not significantly affect the compressive strength, creep, and shrinkage of RAC (De Vries, 1993; Limbachiya, et al., 2000; Dhir, et al., 1999; Gomez-Soberon, 2003).

It is hard to deny that the properties of recycled aggregates are highly dependent on the amount of adhered mortar. The other proper possibilities to develop properties of recycled aggregates are the method of crushing parent concrete and the irregular shape of recycled aggregate. Thus, a few researchers (Sagoe-Crentsil, et al., 2001; Domingo-Cabo, et al., 2009) have interests in creep and shrinkage behavior of recycled concrete. Domingo-Cabo et al. (2009) were observed that shrinkage of recycled concrete had 70 percent higher shrinkage rate than normal concrete after a period of 180 days in the case of using 100% replacement of recycled aggregates and the shrinkage of recycled concrete kept increasing over 252 days. However, Sagoe-

Crentsil et al. (2001) found that the shrinkage of recycled aggregates increased with time and stabilized at about 91 days. The basic reason for the disagreement is that both researchers were not using the same batches such as adding mineral admixtures and using fine recycled aggregates. For creep behavior, Doming-Cabo et al. (2009) found that the substitution percentage of recycled aggregate affected the creep deformations and the creep of recycled concrete was a 51% higher for a period of 180 days than normal concrete.

Katz (2003) reported the effect of partially hydrated waste concrete on the properties of RCA and RAC (made with only coarse RCA). He found a reduction of 25% in the compressive strength of RAC compared to NAC, regardless of the crushing age of the old concrete. Other properties, such as flexural and splitting strengths and drying shrinkage exhibited similar trends. He also pointed out that the effect of RCA on the new concrete properties (strength, elastic modulus, etc.) is similar to the effect of lightweight aggregate on concrete properties.

2.4.6 Water cement ratio

According to available literature, for a given w/c ratio and identical mix proportions, the mechanical and long-term properties of concrete become inferior compared to NAC as the coarse RCA content is increased (Ray & Venkateswarlu, 1991; Limbachiya, et al., 2000; Topcu & Sengel, 2004).

Chinwuba (2011) tested 48 standard concrete cube specimens. Twenty four of the cubes were cast from recycled aggregates while 24 were from virgin aggregates. The results showed that at higher water / cement ratios, the compressive strength of recycled concrete is similar to that of virgin concrete but at lower water / cement ratios, the compressive strength of recycled concrete is appreciably lower than that of virgin concrete. Thus at water / cement ratios of 0.5 and 0.6 the compressive strength ratio of recycled concrete to virgin concrete was respectively 0.89 and 0.985 at crushing age of 28 days and 0.5 and 0.95 respectively at crushing age of 7 days. In addition, the slump and the compacting factor tests revealed that the workability of virgin concrete mix is higher than that of recycled concrete.

Fathifazl, (2008) summarized the properties of recycled aggregate in following table.

Table 2.1: Summery of the studies performed by previous researchers on compressive, tensile and flexural strength of RAC (Fathifazl, 2008)

Investigator	Strength Reduction Using Coarse RCA			Strength Reduction Using Coarse / Fine RCA		
	f'_c	f_t	f_f	f'_c	f_t	f_f
Poon et al. (2004a)	20%↓ and 7-9%↓ [a]	-	-	-	-	-
Poon et al. (2004b)	-	-	-	-	-	-
Santos et al. (2004)	0-15%↓	↓	-	-	-	-
Xiao et al. (2004)	11%↓	-	-	-	-	-
Katz (2004)	-	-	-	-	-	-
Mukherjee et al. (2003)	27%↓	-	-	-	-	-
De Oliveira- Vazquez (1996)	10%↓	-	10%↓	-	-	-
Salem-Burdette (1998)	2%↑-9%↓ [f]	9-14%↓ [g]	-	-	-	-
Ramamurthy-Gumaste(1998)	15-42%↓	-	-	-	-	-
Yamasaki –Tatematsu (1998	↓ [h]	-	-	↓ [i]	-	-
Dhir et al. (1999)	6-18%↓	-	↓	20-30↓	-	↓
Sakata-Ayano (2000)	-	-	-	20%↓	-	-
Nagataki et al. (2000)	14%↑,12% ↑, 9%↓ [j]	-	-	5%↑,9%↑ ,4%↓ [j]	-	-
Limbachiya et al. (2000)	-	-	4%↓-3%↑ [k]	-	-	-
Sagoe-Crentsil, et al.(2001)	≈ 0%	-	-	-	-	-
Ryu (2002a and 2002b)	2-27%↓	-	-	-	-	-
Katz (2003)	--	-	-	25-40%↓	to 46%↓	to 30%↓
Chen et al. (2003)	25-40%↓ l	-	-	-	-	-
Kiuchi-Horiuchi (2003)	8-22%↓	-	-	-	-	-
Shayan-Xu (2003)	28%↓	-	-	-	-	-
Topcu-Sengel (2004)	23.5-33%↓	-	-	-	-	-

[a]For high-strength RAC
[c]bFor low-strength RAC
[c] Different for water or air curing conditions
[d]Higher w/c ratio, lower reduction
[e]Higher w/c, lower difference, Always bearing strength>compressive strength
$\sigma_{br} = \lambda\sigma_c\sqrt{R}$ where λ=0.45, 0.3, and 0.40for conventional concrete, RAC with 100% coarse RCA, and concrete made with combined coarse aggregate containing of 60% of recycled aggregate and 40% of natural aggregate, R is the ratio of bearing area to the

Table 2.2: Summery of the studies performed by elastic modulus RAC (Fathifazl, 2008)

Investigator	Elastic Modulus Reduction Using Coarse RCA%		Elastic Modulus Reduction Using Fine/Coarse RCA%	
	Static	**Dynamic**	**Static**	**Dynamic**
Frondistou (1977)	40%↓	-	-	-
Nishibayashi et al. (1985)	15%↓	-	-	-
Hansen-Boegh (1985)	14-28%↓	14-28%↓	-	-
Sri Ravindrarajah (1987)	-	-	28-32%↓	-
Ong-Ravindrarajah (1987)	6-40%↓	up to 30%↓	16-45↓	up to 30%↓
Loo et al. (1987)	25-35%↓	13-18%↓	-	-
Merlet-Pimienta (1994)	-	-	23-28%↓	-
Topcu-Gunkan (1995)	20%↓	-	-	-
De Oliveira-Vazquez (1996)	25%↓	-	-	-
Salem-Burdette (1998)	16%↓	-	-	-
Nishiura et al. (2000)	2%↓,4%↓ and 2%↑	-	-	-
Kiuchi (2001)	15-20%↓	-	-	-
Corinaldesi et al. (2001)	-	-	-	21%↓
Lamond et al. (2002)	10-33%↓	-	25-40%↓	-
Gomez-Soberon (2002a,b,c)	10%↓	-	-	-
Ajdukiewicz –Kliszewicz (2002)	3-11%↓	-	20-31%↓	-
Gomez-Soberon (2003)	10%↓	-	-	-
Katz (2003)	-	-	37-50%↓	-
Chen et al.(2003)	30%↓[a]	-	-	-
Santos et al.(2004)	14-24%↓	-	-	-
Xiao et al. (2004)	45%↓	-	-	-
Summary	2-45%↓	13-30%↓	20-50%↓	Up to 50%↓

[a] RCA containing brick and tiles

Table 2.3: Summery of the studies performed by previous researchers about shrinkage and creep of RAC (Fathifazl, 2008)

Investigator	Reduction Using Coarse RCA,%		Reduction Using Fine/Coarse RCA,%	
	Shrinkage	Creep	Shrinkage	Creep
Tomozo (1983)	30%↑	-	-	-
Nishibayashi,et al. (1985)	37-110%↑	28-43%↑	-	-
Hansen-Boegh (1985)	50%↑	-	-	-
Sri Ravindrarajah (1987)	-	-	63-98%↑	-
Loo et al. (1987)	59-111%↑[a]	-	49-82%↑[a]	-
Merlet-Pimienta (1994)	-	-	20-46%↑	-
Yanagi et al. (1994)	-	-	17%↑	-
Mesbah-Buyle (1999)	-	-	-	-
Limbachiya et al. (2000)	6-9%↑	33-65%↑	-	-
Han et al. (2001)	20%↑	20%↑	-	-
Buyle-Zaharieva (2002)	80%↑	-	200%↑	-
Lamond et al. (2002)	20-50%↑	30-60%↑	70-100↑	-
Gomez-Soberon (2002a)	20-70%↑ and 263%↑[b]	-	-	-
Gomez-Soberon (2002b)	-	30-47%↑	-	-
Gomez-Soberon (2003)	20-70%↑ and 263%↑[b]	30-47%↑	-	-
Summary	6-111%↑	20-65%↑	17-200%↑	-

[a] Different for water or air curing conditions after 90 days
[b]20-70%↑ in shrinkage for up to 60% RCA replacement and 263%↑ for 100% RCA replacement

2.5 FACTORS AFFECTING SHEAR BEHAVIOR

Shear strength of the beam is controlled by the presence of longitudinal reinforcement, coarse aggregate size, depth of the member, web reinforcement, the presence of axial loads, the tensile strength of the concrete, and shear span to depth ratio (a/d). Some of these parameters are included in design equations and others are not. Web reinforcement, typically called stirrups, is used to increase the shear strength of concrete beams and to ensure flexural failure. This is necessary due to the explosive and sudden nature of shear failures, compared with flexural failures which tend to be more ductile. Web reinforcement is normally provided as vertical stirrups and is spaced at varying intervals along a beam depending on the shear requirements. Alternatively, this reinforcement may be provided as inclined longitudinal bars. Shear reinforcement has very little effect prior to the formation of diagonal cracks.

2.5.1 Shear Span to Depth Ratio

The shear span to depth ratio (a/d) does not considerably affect the diagonal cracking for values larger than 2.5. The shear capacity increases as the shear span to depth ratio decreases. This phenomenon is quite significant in deep beams ($ad/\leq 2.5$) because a portion of shear is transmitted directly to the support by an inclined strut or arch action. For deep beams, the initial diagonal cracking develops suddenly along almost the entire length of the test region (Wight & MacGregor, 2012)

The shear strength is decreased with the increase of shear span to depth ratio. Aly et al. (2015) studied the structural behavior of the concrete beams with recycled aggregate concrete. Sixteen beams were cast to investigate the effect of RCA ratios, the shear span to depth ratios and the effect of different locations and reinforcement of openings on the shear behavior of the tested specimens. All these beams were designed to fail in shear. They found the ultimate shear strength of beams with RCA is very close to those with natural aggregates indicating the possibility of using RCA as a partial replacement to produce structural concrete elements.

2.5.2 Compressive Strength of Concrete

Researchers have concluded that axial compression serves to increase the shear capacity of a beam while axial tension greatly decreases the strength. As the axial compressive force is increased, the onset of flexural cracking is delayed, and the flexural cracks do not penetrate as far as into the beam (Wight & MacGregor, 2012).The tensile strength of the concrete (fct) also affects the shear strength. Because of the low tensile strength of the concrete, diagonal cracking develops along planes perpendicular to the planes of principal tensile stress. The shear strength of an RC beam increases as the concrete material strength increases. The tensile strength of the concrete is known to have a great influence on the shear strength, but the concrete compressive strength ($f'c$) is used instead in most shear strength formulas. This approach is used because tensile tests are more difficult to conduct and usually show greater scatter than compression tests.

The shear strength is increased with the increase of compressive strength of concrete

2.5.3 Longitudinal Reinforcement Content

The longitudinal reinforcement ratio (ρ) affects the extent and the width of the flexural cracks. If this ratio is small, the flexural cracks extend higher into the beam and open wider. When the crack width increases, the components of shear decrease, because they are transferred either by dowel action or by shear stresses on the crack surfaces.

It was found that steel ratio has a significant influence on the shear capacity of RC beams (Hossain, 1984 and Habibullah, 1967) the diagonal shear capacity of concrete is increased with the increase of steel ratio

2.5.4 Maximum Size of Coarse Aggregate

The coarse aggregate type and size noticeably affect the shear capacity, especially for beams without stirrups. The lightweight aggregate has a lower tensile strength than the normal aggregate. The shear capacity of a concrete beam with no stirrups is directly related to the tensile strength, therefore, the failure due to mortar cracking, which is more desirable, could be preceded by aggregate failure instead. The aggregate size also affects the amount of shear stresses transferred across the cracks. Large diameter aggregate increases the roughness of the crack surfaces, allowing higher shear stresses to be transferred ((Wight & MacGregor, 2012).

2.5.5 Width of the Beam

The size of the beam affects the shear capacity at failure. If the overall depth of a beam is increased, it could result in a smaller shear force at failure. The reasoning is that when the overall depth of a beam increases, so do the crack width and crack spacing, causing loss of aggregate interlock. This condition is known as a size effect. The shear strength has no effect on changing the width of beam ((Kani, 1966)

2.6 SHEAR STRENGTH OF RAC

Comprehensive research has been done on both the fresh and hardened properties of recycled aggregate concrete (RAC), but limited, and often contradictory, research has been performed on the structural behavior of RAC. The early research on the structural performance of RAC was published in Japan (Kikuchi, et al., 1994).

Gonzalez & Martinez (2007) tested eight beams with 3% longitudinal reinforcement ratio and 50% recycled coarse aggregate and with different amount of shear reinforcement in their study. The recycled aggregate were obtained from a single source. The beam had a cross-section of 200*mm* x 350*mm*, and they were tested with a span-to-depth ratio of 3.3 as shown in **Figure 2.2**.

The testing setup maintained two spans for each beam, where each span had a different shear reinforcement (one of them had stirrups of 6*mm* or no shear reinforcement and the other had stirrups of 8*mm*). As expected, the flexure cracks started to appear near the center of the beams. Then, flexure cracks appeared away from the center until one of them propagated as a diagonal crack. The beams without shear reinforcement failed abruptly after the diagonal crack extended towards the loading point, while the beams with shear reinforcement showed more load-carrying capacity. The test results were compared to the prediction of Response-2000 software package -which is based on the MCFT- and with various codes, such as the Spanish code (EHE), Canadian code (CSA 23.3), American code (ACI 318) and the Australian code (AS3600). All the predictions were conservative and thus the codes used are feasible to predict shear strength of recycled aggregate beam. They reported that the equations of the codes gave closer results when the beams were without shear reinforcement. On the other hand, the MCFT gave better predictions for the beams with shear reinforcement.

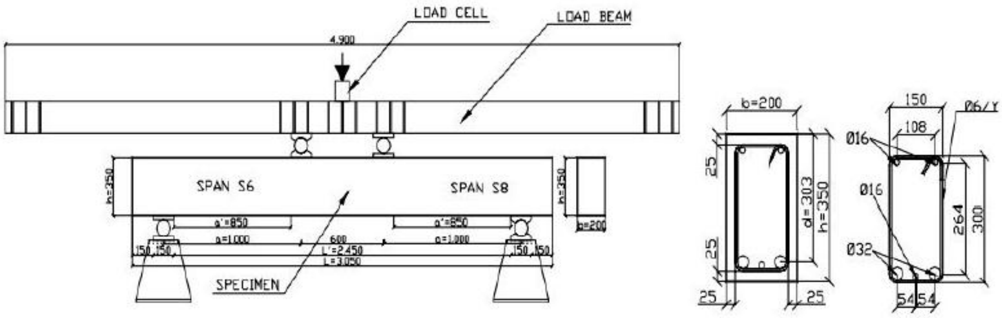


Figure 2.2: Typical beam and testing setup of Gonzalez et al.

Results of their study showed that in terms of both deflection and ultimate shear strength, no significant difference was observed between the RAC and conventional concrete (CC) beams, but they observed notable splitting cracks along the tension reinforcement. They concluded that existing code provisions for shear can be used for the RAC beams.

They repeated the previous study except for adding 8% silica fume to the mix designs. They observed that notable splitting cracks along the tension reinforcements were mitigated by the addition of silica fume.

Ajdukiewicz & Kliszczewicz (2007) conducted a comparative study between beams and columns made from either recycled aggregate (RA) or natural aggregate (NA), both of them coming from different sources. They tested 16 series of beams, each one of them consists of three beams. The beams in each series contain the same amount of reinforcement, but each beam is prepared using different percentage of RA. The 16 series differ in three characteristics; the source of the RA (i.e. river gravel, crushed granite and crushed basalt), the combination and the type (natural or recycled) of fine and coarse aggregates and the concrete compressive strength (low, medium and high). The cross-section and the longitudinal view of the beams are presented in **Figure 2.3**.

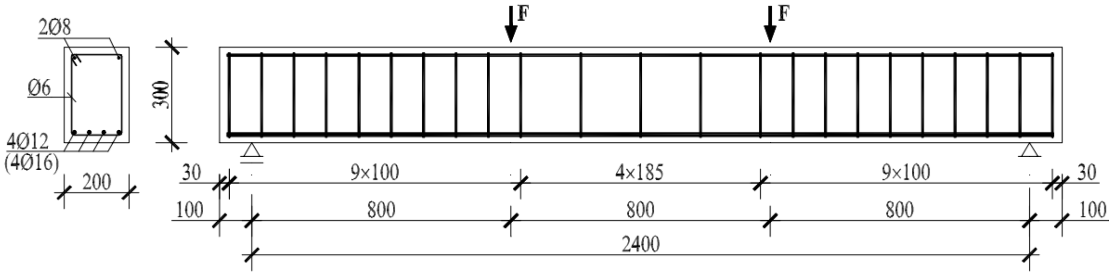


Figure 2.3: Typical Beam dimensions and detailing

As expected, the flexure-type beams failed due to yielding of reinforcement steel with minor damage in the compressed concrete. Also, with the increase of replacement percentage, the modulus of elasticity decreases and deformability of a beam increases. On the other hand, the beams that were designed to study shear had the initial cracks appearing due to flexure, but then after, the shear cracks started to appear till the shear failure happened as appears in **Figure 2.4**.

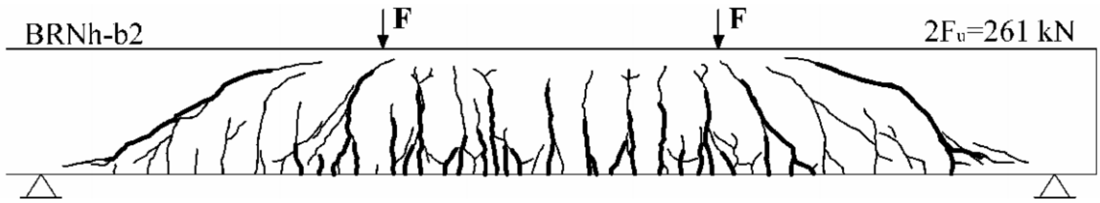
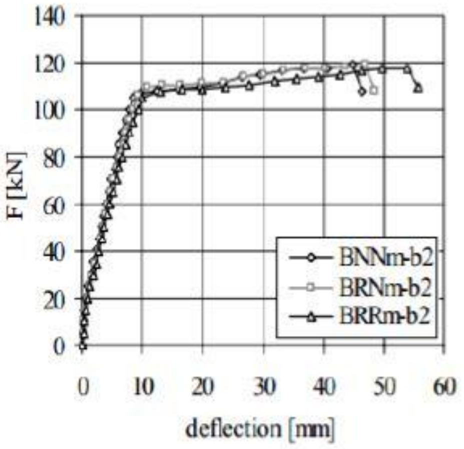
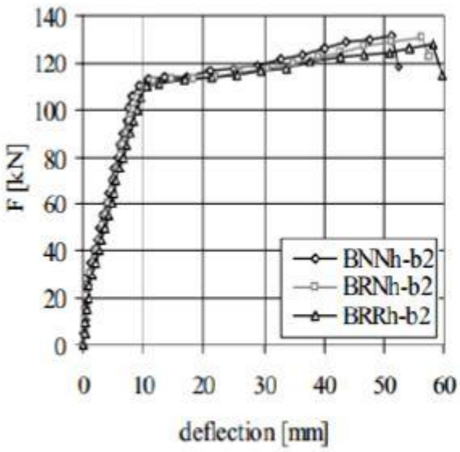


Figure 2.4: Typical shear failure and the cracks propagation

The load-deflection relationships of two beam series in shear up to failure are shown in **Figure 2.5**. The first series (*m*) refers to medium strength concrete beams, whereas the second series (*h*) denotes high strength. To understand the labeling of the beams, *B* stands for basalt as the source of the aggregates, the first and the second *N*'s indicates that the fine and the coarse aggregates, respectively, are natural. If *R* is used in lieu of *N* that means the aggregate is recycled. The *b* indicates that this is a beam (not a column); while number *1* means that the bottom reinforcement of the beam is (*4Ø12*) as opposed to number *2* which means that the bottom reinforcement of the beam is (*4Ø16*). As can be noticed in **Figure 2.5**, the authors' conclusion was that the difference observed in the behavior of beams made from RA and NA beams is insignificant regardless to the replacement ratio. Hence, it is possible to use good quality RA, both fine and coarse, in structural members; however, the serviceability should be taken in consideration.



(a) Medium-strength beams



(b) High-strength beams

Figure 2.5: Load-Deflection relationships of two beam series

Maruyama et al. (2004) tested beams with different longitudinal reinforcement ratios ranged between 2.4% and 4.2%. They also investigated three different water/cement ratios, w/cm, (0.30, 0.45, and 0.60) for their mix designs. They reported that the crack patterns and failure modes of the RAC beams were identical with the conventional concrete (CC) beams. The RAC beams without stirrups showed 10–20% lower shear strength compared with the CC beams.

Arezoumandi et al. (2014) conducted a study on the shear strength of full-scale beams constructed with 100% recycled concrete aggregate (RCA) as well as conventional concrete (CC). They casted six beams using convectional concrete with the help of local ready-mix supplier. Those same beams were crushed, after being tested, to produce the RA that were used in the RA concrete beams for results comparison purposes. The compressive strength of both concrete types was taken as the average of three standard cylinders. Also, splitting tensile strength and flexure strength were tested and all the results are presented in **Table 2.4**. All the beams had a cross-sectional area of (300*mm* x 460*mm*) and length of 4300 *mm*. The targeted compressive strength of both mix designs was 35 *MPa*. To study the effect of the longitudinal reinforcement ratio (r) on shear strength, they used three different values; = 1.27%, =

2.03% and = 2.71%. However, all the beams were without transverse reinforcement within the test regions, the beams detailing are shown in **Figure 2.6**.

Table 2.4: Fresh and hardened concrete results (Arezoumandi, et al., 2014)

Property	CC-1	CC-2	RAC-1	RAC-2
Slump (mm)	140	205	210	130
Air content (%)	8.5	9.0	6.5	5.0
Unit weight (kg/m^3)	2330	2340	2180	2220
Compressive strength* (MPa)	37.2	34.2	30.0	34.1
Split tensile strength* (MPa)	3.48	2.97	2.55	2.65
Flexural tensile strength** (MPa)	3.45	2.90	2.81	2.76

The testing of beams was done in a load frame that has two 490-kN servo-hydraulic actuators to apply load at two points on the beam. The load was applied with displacement control method using 0.5 *mm*/minute load.

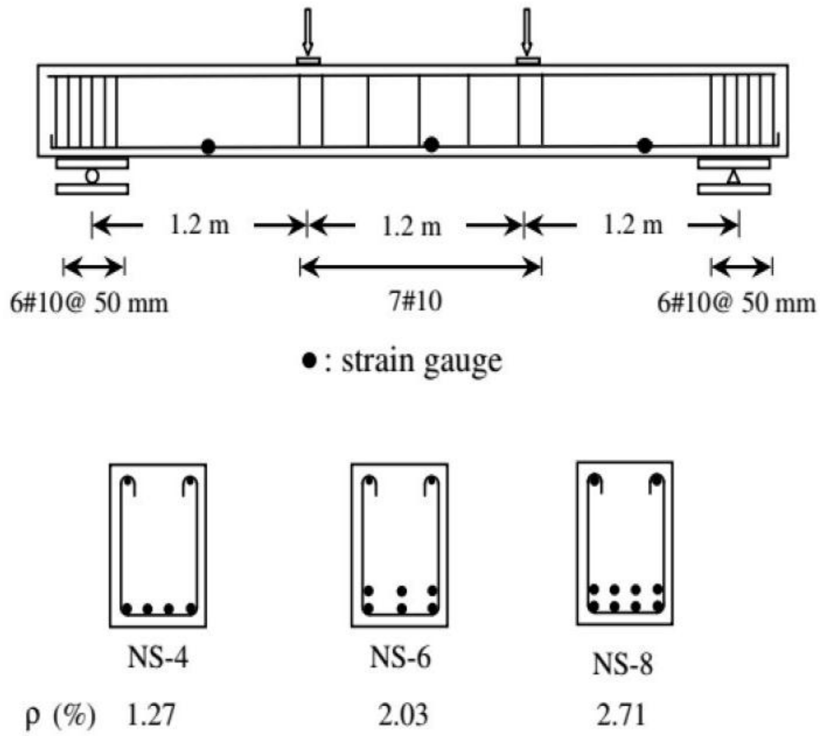


Figure 2.6: Beams detailing and test setup (Arezoumandi, et al., 2014)

The test results are presented in **Table 2.5**. The deflection at mid-span results showed that the beams behaved elastically till the first flexure crack occurred. Increasing the load forced the beams to develop flexure-shear cracks. It is noted that the propagation of cracks in both CC and RA concrete was similar. The beams with higher reinforcement ratio had higher capacity, as expected. They attributed that to the increase in the dowel action and tightened cracks which led to higher aggregate interlock. The shear strength results of the beams were compared to the shear strength predicted from various codes as shown in **Table 2.6**. The range of V_{test} /V_{code} for the CC was from 0.80 to 1.54, while it was 0.76 to 1.38 for the RA concrete. It is noted that the ratio of the RA is lower than the ratio of CC. Furthermore, the author used fracture mechanics approach to predict the shear strength. They used three different equations; Bazant equation (Bazant & Yu, 2005), Gastebled equation (Gastebled & May, 2001) and Xu equation (Xu, et al., 2012).

Table 2.5: Test results summery (Arezoumandi, et al., 2014)

Sections		f'_c (MPa)	V_{test}^* (kN)	$v_{test} = V_{test}/b_w d$ (MPa)	$v_{test}/\sqrt{f'_c}$
CC					
NS-4	1	37.3	121.2	1.0	0.16
	2	34.2	129.9	1.1	0.18
NS-6	1	37.3	143.2	1.3	0.21
	2	34.2	167.0	1.5	0.25
NS-8	1	37.3	173.5	1.5	0.25
	2	34.2	170.8	1.5	0.26
RAC					
NS-4	1	30.0	114.8	0.9	0.17
	2	34.1	113.0	0.9	0.16
NS-6	1	30.0	143.2	1.3	0.23
	2	34.1	124.1	1.1	0.19
NS-8	1	30.0	131.4	1.2	0.21
	2	34.1	140.3	1.2	0.21

Table 2.6: V_{test} /V_{code} for the selected codes (Arezoumandi, et al., 2014)

Section		AASHTO	ACI	AS-3600	CSA	Eurocode 2	JSCE
CC							
NS-4	1	0.82	0.98	0.97	0.80	0.90	1.09
	2	0.95	1.10	1.07	0.93	0.99	1.21
NS-6	1	0.94	1.24	1.02	0.92	0.96	1.16
	2	1.23	1.51	1.23	1.20	1.15	1.39
NS-8	1	1.11	1.50	1.12	1.09	1.16	1.28
	2	1.13	1.54	1.14	1.11	1.17	1.29
Ave.		1.03	1.31	1.09	1.01	1.05	1.24
COV (%)		14.7	18.2	8.4	14.8	11.5	8.5
RAC							
NS-4	1	0.85	1.04	0.99	0.83	0.91	1.11
	2	0.78	0.96	0.93	0.76	0.86	1.05
NS-6	1	1.05	1.38	1.10	1.03	1.03	1.25
	2	0.81	1.12	0.91	0.79	0.85	1.04
NS-8	1	0.84	1.27	0.92	0.83	0.94	1.04
	2	0.86	1.27	0.94	0.85	0.97	1.06
Ave.		0.87	1.17	0.96	0.85	0.93	1.09
COV (%)		11.0	13.6	7.4	11.2	7.2	7.5

Table 2.7 shows the latter results comparison. It was found out that Xu equation (Xu, et al., 2012) gave the best accuracy. They also, compared the test results with the modified compression field theory (MCFT) which is incorporated in several codes such as the AASHTO LRFD-10 and CSA 23.3-04. The ratio V_{test} /V_{MCFT} shows that the MCFT method underestimates the shear strength of all beams. Also, the load-deflection behavior shows good agreement when compared between the experimental results and the MCFT results. They concluded that all the methods that were used to predict shear strength in their study are feasible and can be applied on beams made with 100% recycled aggregate.

Table 2.7: Comparison with fracture mechanics and MCFT

Section		V_{test}/V^{a}	V_{test}/V^{b}	V_{test}/V^{c}	V_{test}/V_{MCFT}
CC					
NS-4	1	1.20	1.05	1.02	1.08
	2	1.32	1.16	1.13	1.19
NS-6	1	1.26	1.23	1.20	1.17
	2	1.51	1.48	1.45	1.48
NS-8	1	1.37	1.45	1.40	1.32
	2	1.38	1.47	1.44	1.33
Ave.		1.34	1.31	1.27	1.26
COV (%)		8.1	14.2	14.5	11.3
RAC					
NS-4	1	1.21	1.07	1.06	1.09
	2	1.14	1.01	1.00	1.02
NS-6	1	1.35	1.33	1.32	1.25
	2	1.12	1.10	1.08	1.01
NS-8	1	1.11	1.18	1.18	1.05
	2	1.14	1.21	1.19	1.08
Ave.		1.18	1.15	1.14	1.08
COV (%)		7.6	10.0	10.0	8.1

They extended their previously mentioned study in another paper (Arezoumandi, et al., 2015) discussing the RA replacement ratio on the shear strength of RC beams. In addition to the 100% RA mix and the NA mix, they used 50% RA mix. The experimental results were compared to the results obtained from the methods mentioned in the earlier paper. The conclusions were that the shear strength of beams made with 100% RA is less by 11% than the beams made with NA. However, beams made with 50% RA gave similar shear strength to the beams made with NA. Nevertheless, all the methods that were used were able to predict the shear strength for all beams with conservancy regardless to the RA replacement ratio.

In recent study, (Knaack & Kurama, 2014) studied the flexure and shear behavior of 12 twin pairs of normal strength concrete beams. They used the direct volume replacement method (DVR) to produce two mix designs; R = 50%, in addition to the conventional concrete mix R =0%. The RA source was foundations of 1920s plant. The targeted compressive strength was 40 *MPa* with (*w/c*) ratio = 0.44. The casted beams were 2.0m length with a cross-section area of 150*mm* x 230*mm*, reinforced with either critical flexure or critical shear reinforcement as shown in **Figure 2.7.** The beams were loaded monotonically until their failure using four-points test setup with loading rate of 2.5 *mm*/minute, see Figure 2.7. During the test, the initial cracks and their propagation were observed. For the shear critical-section beams, the initial cracks were due to flexure since the beams were slender. Then, diagonal cracks started to appear and propagate causing a sudden failure when the shear strength was reached. The results showed that increasing R% had more reduction effect of the initial stiffness of RA beams than on their flexure and shear strength.

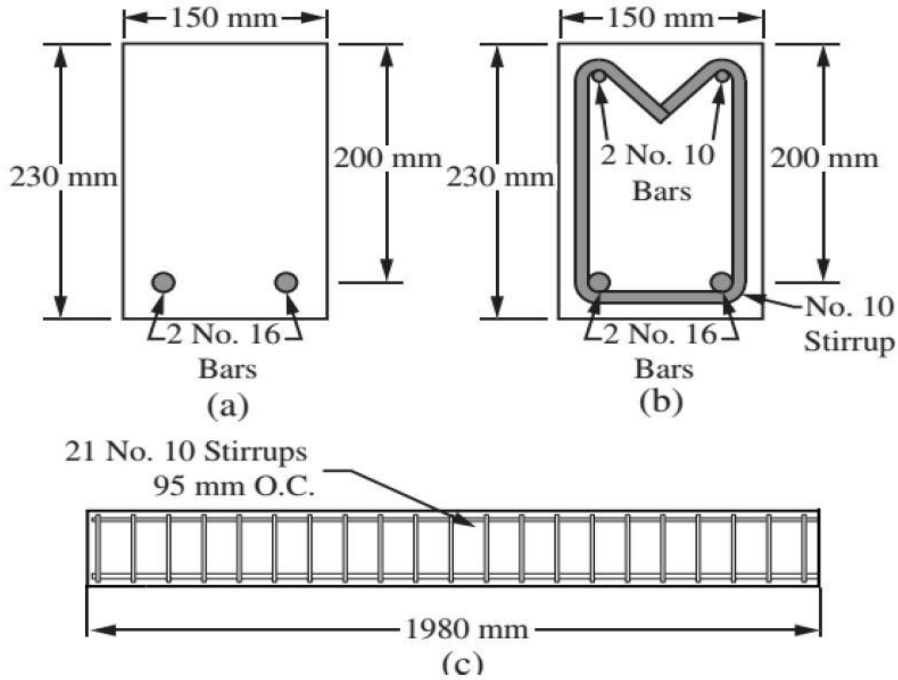


Figure 2.7: (a) Shear-critical section (b) Flexure-critical section (c) Beam (Knaack & Kurama, 2014)

Furthermore, in their efforts to predict the behavior of the beams, they used DRAIN-2DX software package (Prakash, et al., 1993) to model the beams. They also did the convectional analysis using ACI-318. Comparing all the results (i.e. experimental, from DRAIN-2DX and the ACI-318 analytical results), a good agreement was found among them as shown in **Figure 2.8**. Hence, they concluded that the existing design standards are applicable for the design of RA concrete. An interesting note by the authors was that even though each pair of the beams was saw-cut in half from longer beam, the load-deflection behavior was different between them. That renders how the concrete material is so complex and shows the inherit variability in the behavior of concrete members

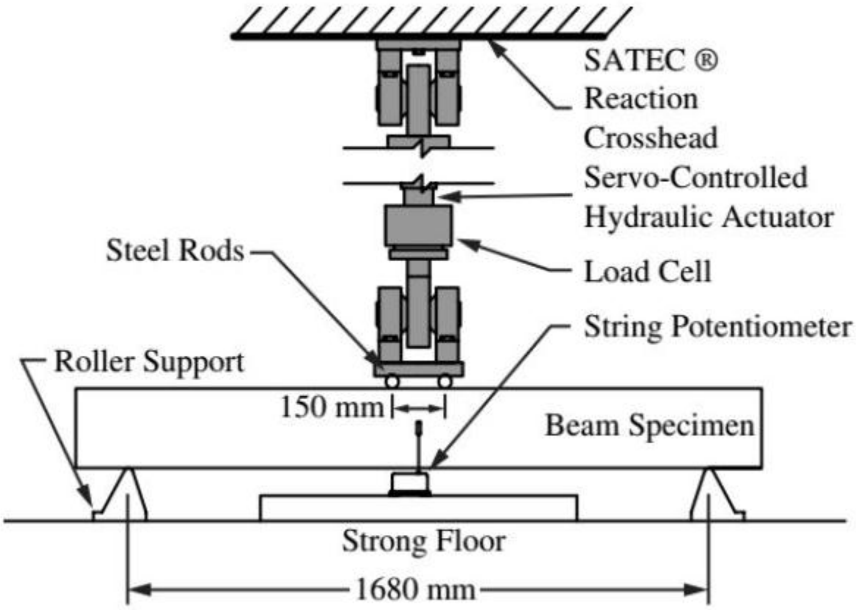


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Figure 2.8: Test setup (Knaack & Kurama, 2014)

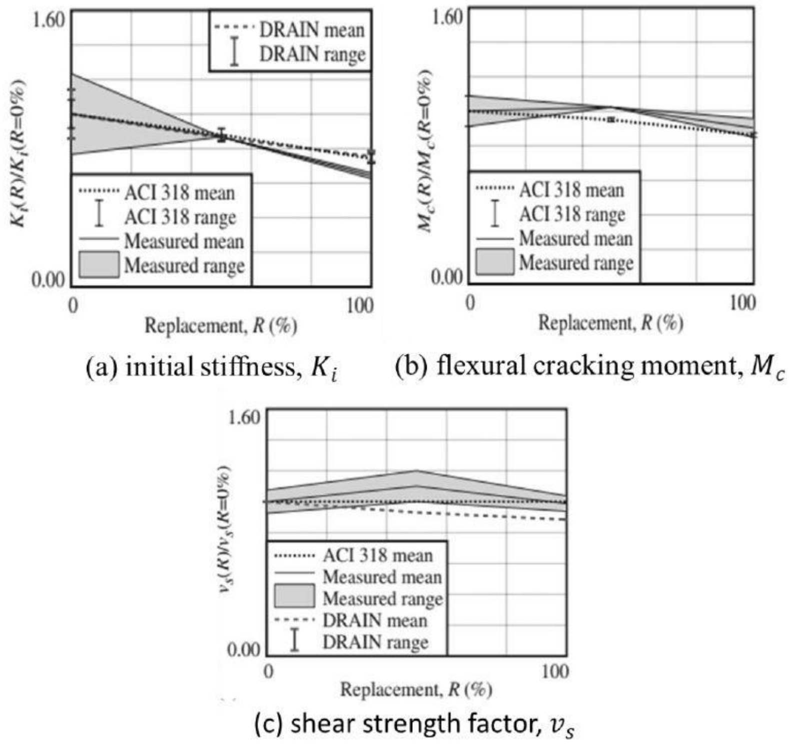


Figure 2.9: Effect of RCA on shear-critical behavior (Knaack & Kurama, 2014)

Choi et al. (2015) evaluated the shear strength of 20 reinforced concrete beams with different span-to-depth ratios (1.50, 2.50, and3.25), longitudinal reinforcement ratios (0.53%, 0.83%, and 1.61%),and RCA replacement ratios (0%, 30%, 50%, and 100%). Results of their study showed that the shear strength of the RAC beams was lower than that of the CC beams with the same reinforcement ratio and shear span-to-depth ratio. They reported that beams with smaller span-to-depth ratios and a higher percentage of recycled aggregate showed a higher reduction in shear strength.

Rahal & Alrefaei, (2015) did an experimental study in which they used different RA replacement ratios and investigated their effect on the shear strength of RC beams. They tested five beams with target compressive strength of 50 *MPa*. The RA replacement ratios were 0%, 10%, 20%, 35% and 100%. The detailing of the beams is shown in **Figure 2.10.**

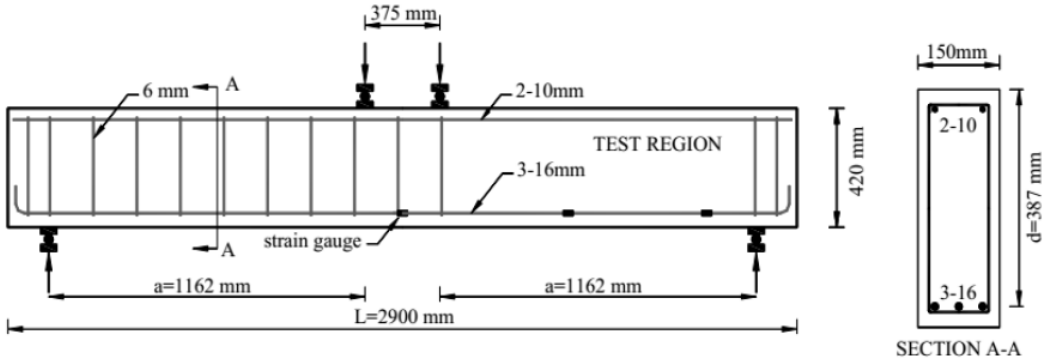


Figure 2.10: Detailing of the beams

The shear strength was normalized with respect to the compressive strength of each beam. The results showed that the shear strength in RA concrete beams was in fact higher than the control beam. But, they also found that the modulus of elasticity was reduced in the RA beams by up to 14%. However, they recommended that these conclusions should be interpreted carefully with respect to the testing variations. The results are presented in **Figure 2.11** and **Figure 2.12**

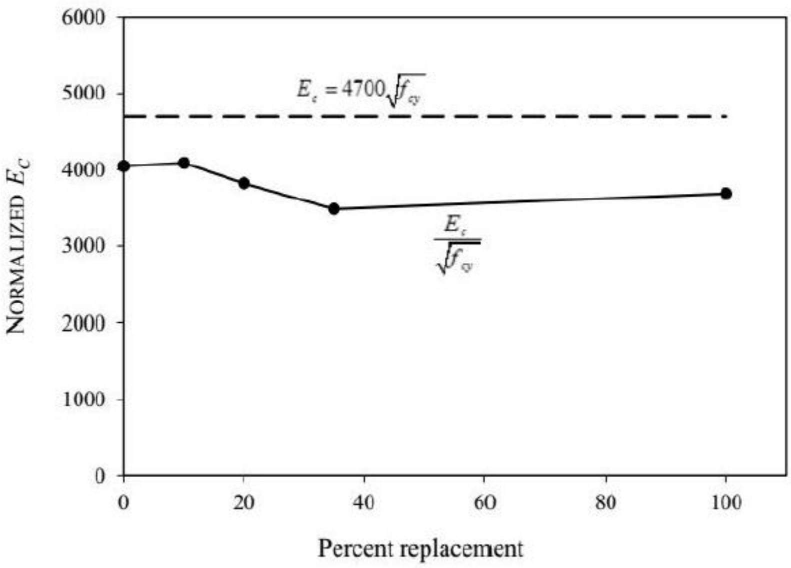


Figure 2.11: Effect of replacement ratio on modulus of elasticity

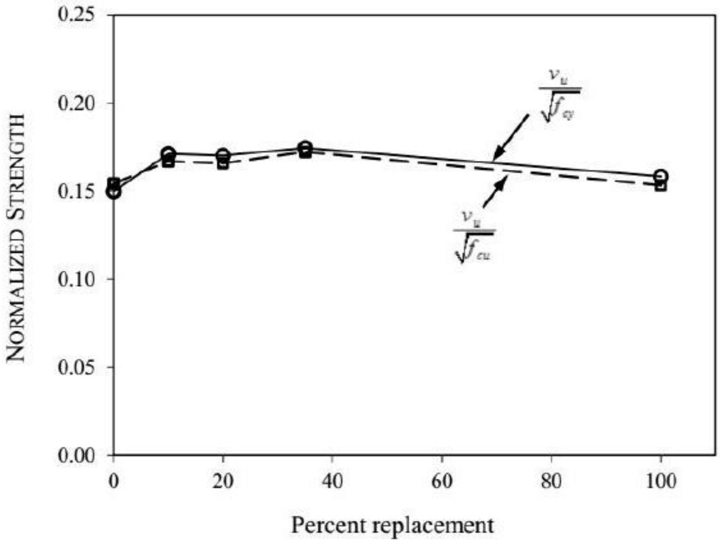


Figure 2.12: Effect of replacement ratio on normalized shear strength

Sang-Woo, et al., (2013) found similar results in their study. They tested 15 beams made with different replacement ratios. To study the size effect on the beams, their width was varied from 200*mm* to 400*mm* and their effective depth from 300*mm* to 600*mm*. Their results showed that the change in the width had no size effect on the beams made with RA; however, the shear strength decreased with the increase of the effect depth regardless to the replacement ratio. More importantly, they found out that the shear reduction and the crack pattern in RA beams were similar to those made with NA.

Also, similar conclusions were reached in a study that was done by (Deng, et al., 2013), in which they considered nine beams with three different coarse aggregate replacement ratios equal to 0%, 50% and 100%. They found out that the shear strength is slightly affected in the RA beams when compared to NA beams, if the compressive strength among the beams was maintained constant among the beams.

Xiao et al. (2012) tested 32 shear push-off specimens with different percentages of recycled coarse aggregate replacement. They reported no significant difference observed in terms of shear stress-slip curves, crack propagation path and shear transfer performance across cracks between the RAC and CC specimens. They also concluded that recycled aggregate replacement up to 30% did not affect ultimate shear load, but for higher percentages of RCA replacement, the ultimate shear load decreased.

Fathifaz, et al., (2009) published a paper about the shear strength of RC beams made with RA and without stirrups. In that paper, they argued that the reduction in the strength and properties of RC beams made with recycled aggregate is not inherited. In their opinion, the cause of the reduction is due to the use of conventional method of mixing, which replaces the amount of the natural aggregate in a mix with recycled aggregate directly. Those methods of mixing neglect the residual mortar around the recycled aggregates, causing the concrete made with RA to have more mortar when the new mortar is added compared to the NA concrete. In other words, the shear plane go through less aggregate and more mortar in the RA concrete due to the presence of the residual mortar. So, the lack of aggregate in RA concrete is the reason behind the reduction in shear strength reported by some researchers. To validate their proposition, they made two types of concrete mixes. The first is a mix made with natural aggregate and proportioned as per ACI code method. The second is a mix made with RA and the proportions were calculated using their proposed method which is called Equivalent Mortar Volume (EMV). They used both limestone (63.5% recycled aggregate replacement) and river gravel (74.3% recycled aggregate replacement) as coarse aggregate for their mix designs. They tested beams with four different shear span-to-depth ratios ranging between 1.5 and 4, and also with four different effective depths (250, 375, 450, and 550 mm) to investigate size effect. Twenty beams were tested and all of them had no shear reinforcement. They concluded that the failure mode was very similar in both RA concrete and NA concrete. Moreover, using EMV method increases the aggregate interlock mechanism and consequently the shear resistance. They reported superior shear strength for the RAC beams. The international building codes, particularly the ACI-318, the CSA23.3-04 and Eurocode 2, can be used to predict the shear strength of RC beams made with RA irrespective to (*a/d*) and the depth of the beam.

Schubert, et al., (2012) studied 14 slabs (0.2 x 0.5 x 2.3 m) with 100% recycled coarse aggregate without shear reinforcement under four-point load condition. A section of the slabs is shown in Figure 2.13 where the width was 500*mm*. They compared the experimental results to various models, such as Swiss Standard, Eurocode 2. The results indicated that the shear strength of RC beams made with RA are promising and very close to those made with NA, and that it can be predict using the utilized building standards.

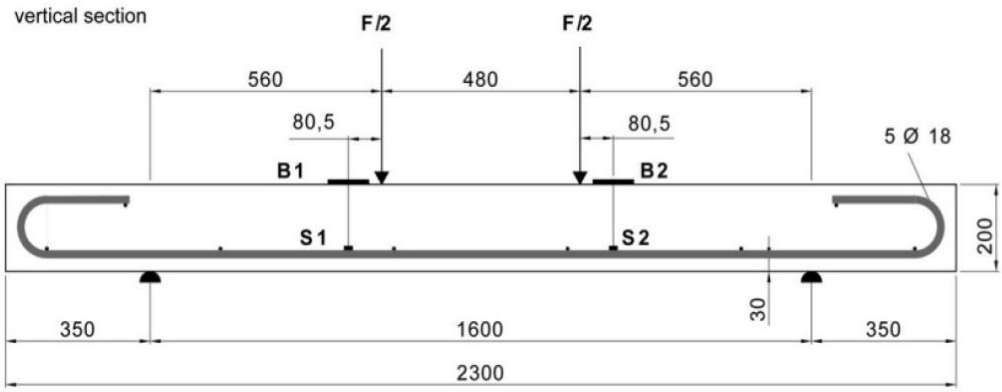


Figure 2.13: Slab section (Schubert, et al., 2012)

Some studies reported that the RA that were obtained from high strength concrete, can be used to produce new concrete that have shear strength comparable or superior to concrete made with NA. To examine these reports, Lian, et al., (2013) experimentally studied beams made with 25% RA along with beams made with NA. The recycled aggregate were from a crushed concrete of strength 25-30 *MPa*. They made three beams from each concrete mix with dimensions of 150*mm* x 200*mm* x 1200*mm*. The typical beam detailing and the testing setup are shown in **Figure 2.14**. The shear span-to-depth ratio varied in the tested beams, namely *a/d* =1.0, 1.5 and 2.0.

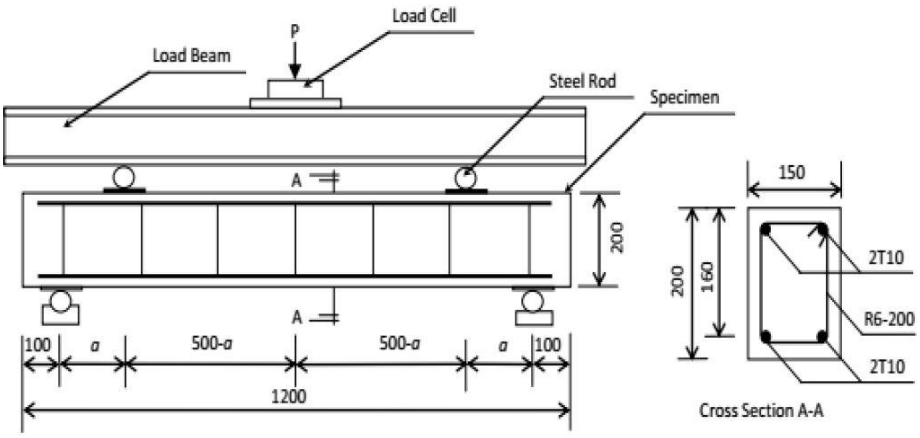


Figure 2.14: Typical beam and test setup (Lian, et al., 2013).

Their results suggested that the RA beams performed very well or even slightly better for *a/d*= 1.5 and 2.0. However, they shear strength was less when *a/d*= 1.0 when compared to the NA beams. Regarding the effect of shear span-to-depth ratio, their conclusions matched the existing literature which proved the decrease in the shear strength when the *a/d* is increased. Finally, upon comparing their experimental results with the existing codes; such as ACI 318, AS3600 and the Eurocode-2, they found that all the predictions were conservative and that the ACI 318 predictions were the closest.

In addition to the considered studies in this chapter, there have been a few other less significant research studies addressing the shear strength behavior of RC beams made with recycled coarse aggregate and subjected to shear. Such studies include the work of (Al-Zahraa, et al., 2011; Ikponmwosa & Salau, 2011; Wang, et al., 2013; Yu & Yin, 2015).

All of these studies were conducted on RC beams made with recycled stone aggregate. Therefore, this study was planned to evaluate the diagonal shear strength of RC beam made with brick aggregate and recycled brick aggregate without shear reinforcement to validate the provisions of shear capacity of RC beams of existing design codes. The test results were compared with the results obtained from RC beams made with brick aggregate. Moreover, Diagonal shear capacity of the beams was evaluated by four-point loading and compared with different codes, such as ACI318-14, AASHTO LRFD, CSA, BS 8110, JSCE, and Model code 2010. Also, the results were compared with corresponding results obtained from equations formulated based on the fracture mechanics approach by Bazant and Yu (2005), Gastebled and May (2001), Xu et al. (2012) Zsutty (1968) and Niwa et al. (1986) Furthermore, the results were compared with the existing shear database (Shilang et al., 2012) results to understand the position of the data points compared to the data points obtained by many researchers.

2.7 SHEAR CAPACITY OF BRICK AGGREGATE

The main subject of this research is the shear behavior of reinforced concrete (RC) beams made with recycled brick aggregate. By exploring the literature on diagonal shear capacity of RC beam, it is found that a large number of investigations was conducted on this topic. The variables investigated were the ratio of longitudinal reinforcement content (Eyþór & and Sigurður, 2011; Hamrat, et al., 2016), width of the beam (Kani, 1966), compressive strength of concrete (Hamrat, et al., 2016), shear span to depth ratio (Hamrat, et al., 2016), type of aggregate (Janaka Perera & Mutsuyoshi, 2013) and maximum size of coarse aggregate (Derek, 2015).

Akhtaruzzaman & Hasnat, (1986) investigate the structural behavior of concrete made with crushed brick as the aggregate. Their investigation involved the testing of forty-eight reinforced concrete rectangular beams made with crushed brick as aggregate and containing no web reinforcement. The beams were tested under four-point loading to investigate shear and flexural strength with the only variables being concrete strength and shear span to effective depth ratio. Concrete beams containing natural aggregate were also tested so that the results could be compared.

The authors recorded a lower value of transitional span to effective depth ratio between diagonal tension failure and flexure failure for brick aggregate concrete beams. This indicates that the brick aggregate concrete beams have a higher shear strength compared to normal weight concrete beams produced with natural aggregate.

They also reported that the difference between the shear strength of brick aggregate concrete beams and normal weight concrete beams is more pronounced when concrete strength is low. This increase in shear strength is due to the higher tensile strength of the material. The difference is about 15 to 35% depending on the concrete strength and the span to effective depth ratio. This crucially means that brick aggregate concrete beams will require less web reinforcement. This coupled with the added advantage of brick aggregate concrete beams having a lower unit weight, make it a suitable structural material with significant economic benefits.

2.8 SHEAR FAILURE MECHANISMS IN BEAMS WITHOUT STIRRUPS

The tensile stresses develop in beams due to axial tension, bending, shear, torsion, or a combination of these forces. Concrete is weak in tension, and the beam will collapse if proper reinforcement is not provided. As the load is increased in such a beam, vertical flexural cracks developed at the section of maximum bending moment when the tensile stresses in concrete exceeded the modulus of rupture of concrete, or $f_r = 7.5\lambda\sqrt{f'_c}$ (Nadim & Al-Manasser, 2015). Shear stresses increase proportionally to the loads. In consequence, diagonal tension stresses of significant intensity are created in regions of high shear forces, the location of cracks in the concrete beam depends on the direction of principal stresses. For the combined action of normal stresses and shear stresses, maximum diagonal tension may occur at about a distance d from the face of the support. Longitudinal tension reinforcement is not such effective in resisting longitudinal tension near the tension face. It does not reinforce the tensional weak concrete against the diagonal tension stresses. Eventually, these stresses attain magnitudes sufficient to open additional tension cracks in a direction perpendicular to the local tension stress which are called as diagonal cracks (web-shear crack), in difference to the vertical flexural cracks. If the inclined crack starts at the top of an existing flexural crack and propagates into the beam, the crack is referred to as flexural-shear crack (**Figure 2.15**). Flexural-shear cracks are the most common type found in reinforced concrete beams. A flexural crack extends vertically into the beam; then the inclined crack forms, starting from the top of the beam when shear stresses develop in that region. In regions of high shear stresses, beams must be reinforced by stirrups or by bent bars to produce ductile beams that do not rupture at a failure. In beams in which no reinforcement is provided to counteract the formation of large diagonal tension cracks, their appearance has far-reaching and detrimental effects. For this reason, it is important to find the methods of predicting the loads at which these cracks will form.

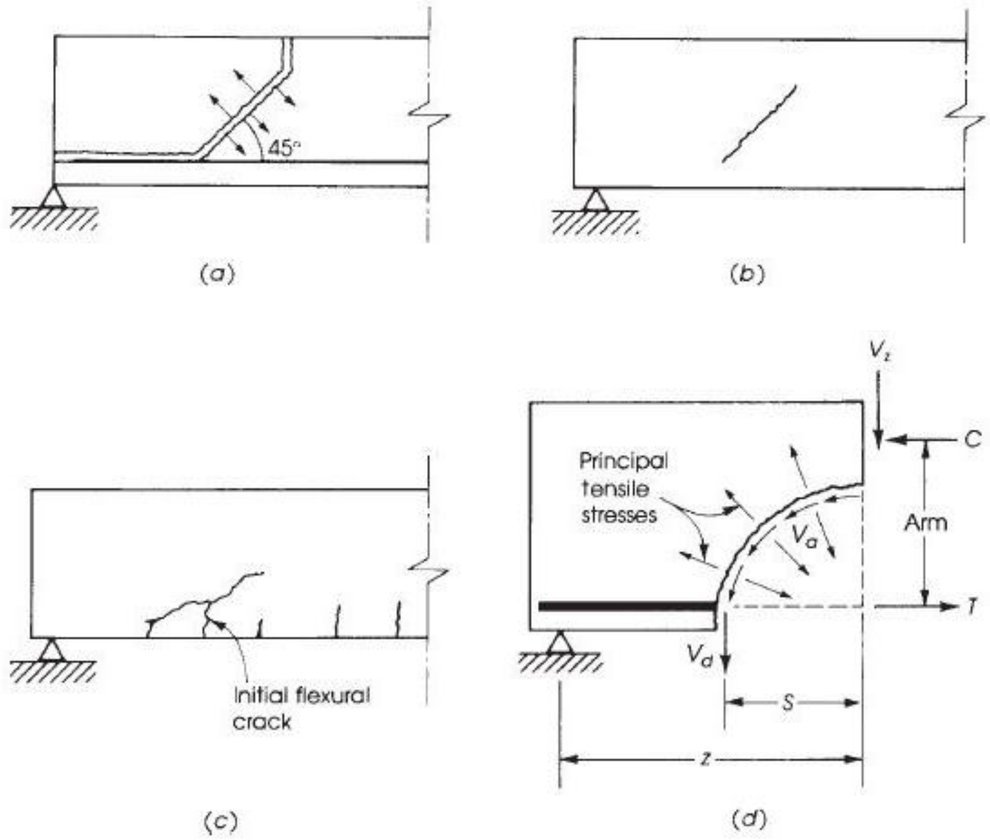


Figure 2.15: Shear failure (a) general form, (b) web-shear crack, (c) flexural-shear crack, (d) analysis of forces involved in shear (source: Nadim et al)

2.9 SHEAR RESISTANCE COMPONENTS

A simply supported beam diagonally cracked in a pure shear region is shown in **Figure 2.16**. It may be seen from the figure the total shear force *(V)* is resisted by three major components (ASCE-ACI, 1973):

• Shear stress in uncracked concrete (V_c)

The contribution of this mechanism can be determined in combination of compressive and shear stresses and the depth of the uncracked zone.

• Interface shear transfer (V_a)

The faces of a crack are generally rough, and as a result of relative motion along the crack surfaces providing the crack to transfer the shear. This form of shear transfer has invariably been referred to as interface shear, shear friction, and aggregate interlock.

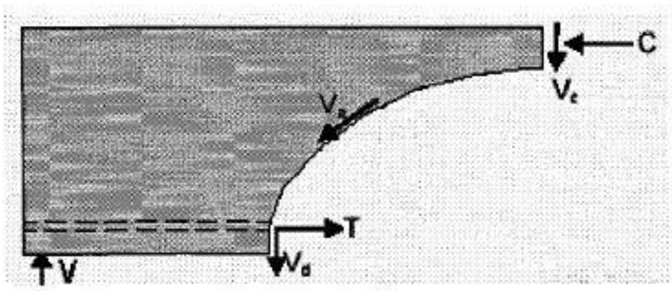


Figure 2.16: Shear resistance component in beams

• Dowel action (V_d)

Dowel action can be developed by three mechanisms: the flexure of the reinforcing bars, the shear strength across the bars, and the kinking of the reinforcement. Dowel action can only occur at the expense of large displacements. Therefore, at acceptable crack displacements, dowel action is not significant.

The contribution of each of these mechanisms varies depending upon the type of member and the relative magnitude of the stresses and the level of loading. For rectangular beams without shear reinforcement, it is reported (ASCE-ACI426, 1973) that after an inclined crack has formed; the proportion of the shear transferred by the various mechanisms is as follows: 15 to 25% by dowel action, 20 to 40% by the uncracked concrete compression zone, and 33 to 50% by aggregate interlock or interface shear transfer.

Among the above-mentioned shear transfer mechanisms, the interface shear transfer in recycled reinforced concrete (RRC) members is of high interest due to the existence of the residual mortar and consequently lower maximum original virgin aggregate size in RAC, which might lead to less rough, and smoother diagonal crack interface, and consequently smaller shear strength.

- *Size Effect*

The size of the beam affects the shear capacity at failure. If the overall depth of a beam is increased, it could result in a smaller shear force at failure. The reasoning is that when the overall depth of a beam increases, so do the crack width and crack spacing, causing loss of aggregate interlock. This condition is known as size effect.

The shear strength of beams without web reinforcement generally decreases as the effective depth increases (Kani, 1966). In particular, the dowel and aggregate interlock components may decrease significantly as the crack width above the main reinforcement tends to increase. However, well distributed longitudinal reinforcements can contribute to size effect reduction (Collins, 1994). Therefore, it can be noticed that the size effect is also of significant interest in RC members due to the role of aggregate interlock action in deeper beams.

2.10 FAILURE MODE IN SHEAR

The various failure modes in shear without shear reinforcement are described in this section.

2.10.1 Diagonal Failure

Many types of structural concrete members other than beams have been reported to fail due to shear distress or diagonal failure e.g. slabs, foundation, columns, corbels, and shear walls. It is believed that the shear transfer mechanism is very similar or the same in all the cases but the cracking pattern may differ. A combination of shearing force and moment is the fundamental cause of diagonal failure (Ziara, 1993).

2.10.2 Diagonal Tension Failure

The diagonal crack initiates from the last flexural crack formed. The failure occurs in beams when the ratio a/d is approximately 2.5 – 6.0 in the shear span “a”. The crack propagates through the beam until it reaches the compression zone. When the beam reaches a critical point it will fail as a result of the splitting of the compression concrete. Often this happens almost without a warning and the failure becomes sudden and brittle **Figure 2.17** (Ziara, 1993).

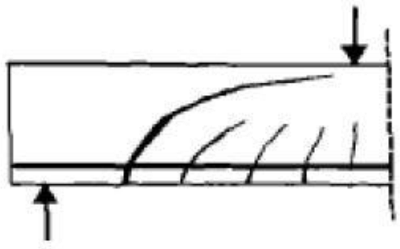


Figure 2.17: Diagonal tension failure

2.10.3 Shear Tension Failure

This type of failure is similar to diagonal tension failure but applies to short beams. The shear crack propagates through the beam but doesn't cause failure of the beam on its own. Secondary cracks travel along the longitudinal reinforcement from the last flexural crack and can cause a loss of bond between the reinforcement and the concrete or anchorage failure **Figure 2.18.** When the beam reaches a critical point it will fail as a result of the splitting of the compression concrete (Ziara, 1993).

Figure 2.18: Shear tension failure

2.10.4 Shear Compression Failure

On the other hand, if the diagonal shear crack propagates through the beam, causing failure when it reaches the compression zone without any sign of secondary cracks as is described in shear tension failure, it's referred to as a shear compression failure **Figure 2.19**. This failure mode applies to short beams. The ultimate load at failure can be considerably more than at diagonal cracking as a result of arch action.

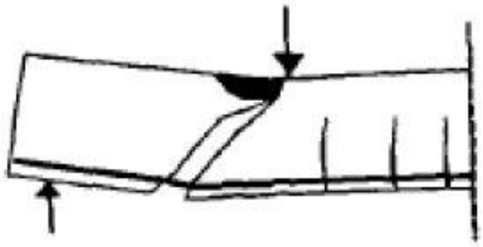


Figure 2.19: Shear compression failure

2.10.5 Flexural Failure

Flexural cracks are mostly moment dependent and in long beams. Consequently, the cracks develop where the maximum moment is in the beam **Figure 2.20**. When the shear stress in the concrete reaches its tensile strength, cracks develop. The cracks are almost vertical and cause failure to the beam due to either of these two cases (Ziara, 1993):

a) Under-reinforced beams: The longitudinal reinforcement yields excessively resulting in failure in the concrete compression zone.
b) Over-reinforced beams: Concrete in the compression zone fails above the flexural crack before the longitudinal reinforcement yields.

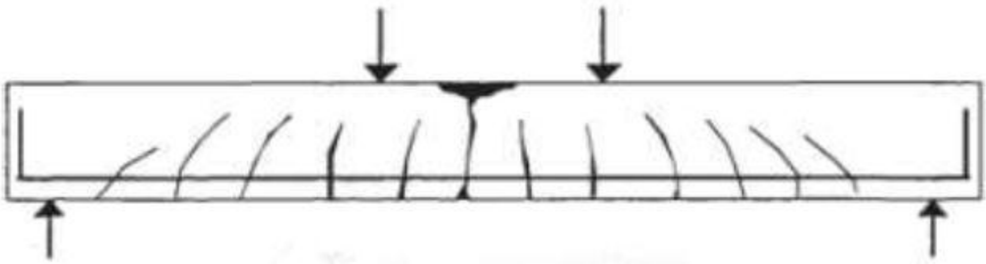


Figure 2.20: Flexural failure

2.10.6 Deep Beam Failure

Deep beams can withstand considerably more load than at diagonal cracking and are considered by many to be the result of arch action, as was mentioned earlier. It can lead to at least the two following failure cases:

2.10.6.1 Anchorage Failure

A slip or a loss of bond of the longitudinal reinforcement can be considered as anchorage failure **Figure 2.21**. It can be linked to dowel action where the aggregates interlocking resistance around the bar has failed resulting in the splitting of the concrete.

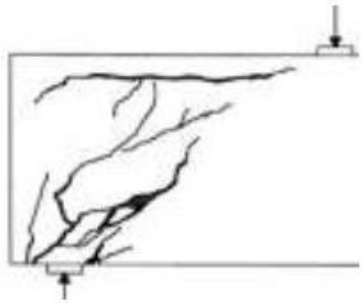


Figure 2.21: Anchorage failure (Hong, et al., 2002)

2.10.6.2 Bearing Failure

Failure at the support is a result of bearing stresses exceeding the bearing capacity of the concrete. If the bearing plate is too small it can result in premature failure of the concrete over the support **Figure 2.22** (Hong, et al., 2002).

Figure 2.22: Bearing failure (Hong, et al., 2002)

2.11 EXISTING STRUCTURES USING RAC

Some countries have successfully employed crushed concrete as a substitute for natural aggregate in the construction of major motorways and other large structures, during the past a few decades. However, the recycling of construction waste was first done at a large scale after the Second World War in Russia and Germany - as it was needed to remove the war-torn buildings and build new ones - construction waste was used as an important resource. Hansen (1986) has extensively reviewed case histories around the world and some of the examples cited by him are briefly recapped as follows. In Belgium, concrete from an old lock wall was crushed and used as new aggregate for construction of a new and larger lock at the same location. In the Netherlands, the first application of RCA was for partition walls in an apartment building, but since then it has been extended to some other projects in highway and airport construction. In Russia, coarse RCA has been used for foundations and fine RCA as mineral filler in asphalt. In Japan, RCA has been used in real structures since 1984. Two small test structures were built by the Building Research Institute of the Ministry of Construction. Also, RCA was used to build the Building Research Establishment (BRE) Cardington Laboratory in the UK, in which over 100 tons of coarse RCA was used in a ready-mix RAC (Digest433., 1998). According to De Vries (1993), coarse NA was replaced by coarse RCA in the foundations and walls of the Caland canal, near Rotterdam-Holland, in order to protect the entrance to the lock on the canal. Collins (1994) reported the construction of a multistory house in Copenhagen in which RCA from demolished houses was used. In Germany, an office building using RCA was built in Darmstadt in 1998 (Xiao, et al., 2012).

Figure 2.23 shows a complex of residential buildings “Waldspirale” in Darmstadt, Germany, described as attractive in terms of architectural form, built in 1998, for which all of the internal structure elements, as well as the base plate, were made of concrete with recycled coarse aggregate.

Figure 2.23: Complex of residential buildings “Waldspirale”, Darmstadt, Germany

CHAPTER 3: EXPERIMENTAL PROGRAM

3.1 GENERAL

The objective of this study was to investigate the shear performance of reinforced concrete (RC) beams composed of RBA. The experimental program consisted of 32 tests (16 beams for each RBA and BA) performed on full-scale RC beams. The principal parameters investigated were: Concrete type – recycled brick aggregate concrete or brick aggregate concrete, the compressive strength of concrete, shear span to depth ratio and amount of longitudinal reinforcement.

Also, as part of this study, small scale testing was performed to determine hardened concrete properties such as compressive strength and splitting tensile strength.

3.2 MATERIALS

3.2.1 Concrete

For this study, two mix designs were produced and evaluated for shear performance. A mix design was used as a baseline for reference throughout the study and also as the parent material for the recycled concrete aggregate. The water-to-cement ratio was 0.50, and the design air content was 2%. The specified amount of fine aggregate as a volume of total aggregates was 40%.

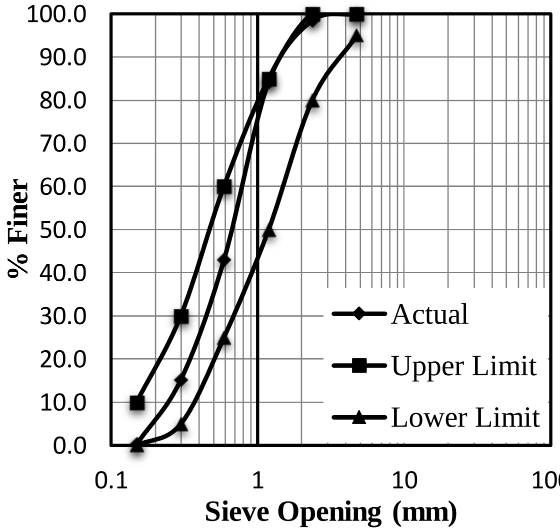
3.2.1.1 Aggregate

For the BA mix, bricks were collected from a local market for coarse aggregate and broken it with a maximum nominal aggregate size of 3/4 in. The recycled aggregate were obtained from a single source. The grading of the aggregate was controlled as per ASTM-C33 (2016). Natural sand was used as fine aggregate. Aggregates were tested for specific gravity, absorption capacity, fineness modulus (FM) and unit weight. Brick and recycled aggregate are shown in **Figure 3.1.** The grading of the aggregates satisfies the requirement of ASTM-C33 (2016) as shown in **Figure 3.2.**

For the RBA mixes, the coarse aggregate consisted of recycled brick aggregate was used. Properties of coarse and fine aggregates summarized in **Table 3.1** and hardened properties of concrete are summarized in **Table 3.2**

With compare to fresh brick aggregate, the abrasion value of recycled aggregate is more. Similarly, unit weight is higher than fresh brick aggregate this is due to adherence of mortar around the brick aggregate **(Table 3.1).**

Figure 3.1: Brick aggregate and recycled brick aggregate respectively



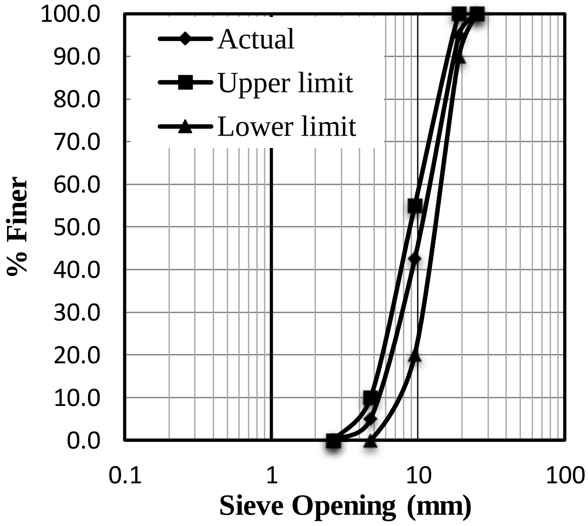


Figure 3.2: Grading Curves of Fine and Coarse Aggregates (Left – Fine Aggregate, Right – Coarse Aggregate)

Table 3.1: Properties of Coarse and Fine Aggregates

Type of Aggregate		Specific gravity	Absorption	Unit weight (SSD) Kg/m^3	Abrasion	FM
Fine Aggregate	Sand	2.46	3%	1574	-	2.58
Coarse Aggregate	Brick chips	1.98	10.5%	1209	39%	6.6
	Recycled aggregate	2.14	22.4%	1204	42%	6.6

Figure 3.3: Specimen for Concrete Property

Table 3.2: Fresh and Hardened Properties of Concrete

Property	Concrete Made with BA		Concrete Made with RBA	
	24 MPa	29 MPa	24 MPa	29 MPa
Slump (mm)	150	125	125	160
Air content (%)	1.9	2.2	2	2.3
*Compressive strength (MPa)	23.7	28.7	24.1	27.5
*Split tensile strength (MPa)	2.5	3.2	2.2	2.8

*Values represent the average of three cylinders

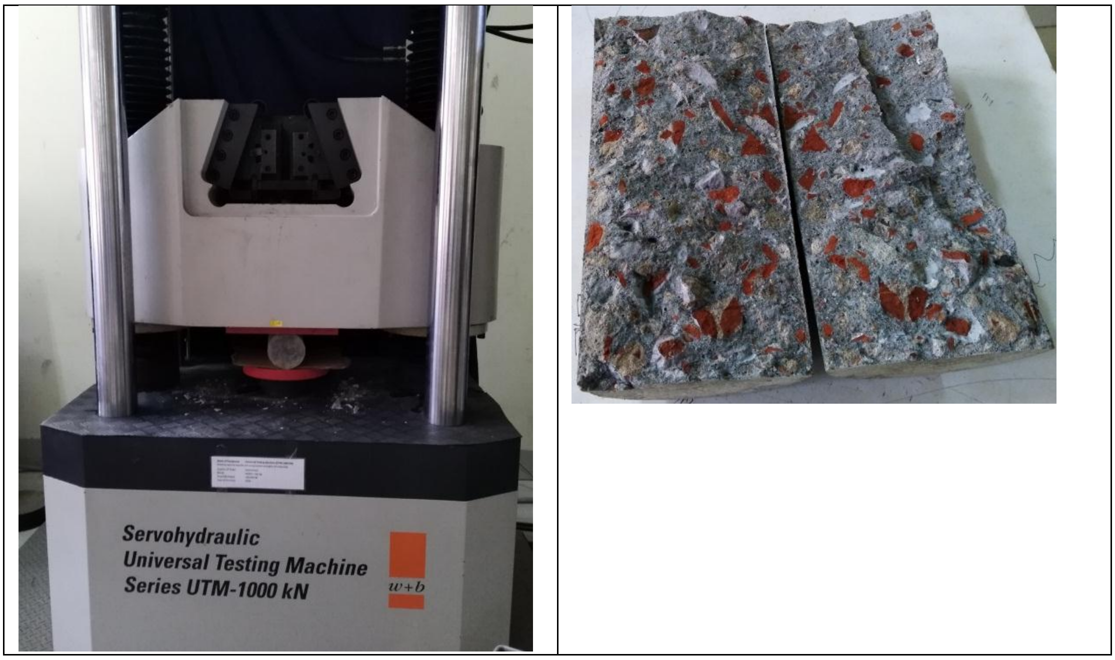


Figure 3.4: Split tensile strength test

3.2.1.2 Cement

CEM Type II B-M cement containing 65-79% clinker, 21-35% mineral admixture (slag, fly ash, and limestone), and 0-5% gypsum was used. As of mixing water, tap water was used.

3.2.2 Reinforcing Steel

Shear reinforcement for the test specimens consisted of A615, Grade 60 #3 reinforcing bars. Longitudinal reinforcement for the test specimens consisted of A615, Grade 60 #5 reinforcing bars. All the steel reinforcement was tested in accordance with ASTM A370 (2017) “Standard Test Methods and Definitions for Mechanical Testing of Steel Products” to obtain the mechanical properties, which are summarized in **Table 3.3**. These results are the average of three replicate specimens. Rebar testing in UTM machine and typical stress-strain curve of rebar are shown **Figure 3.5**

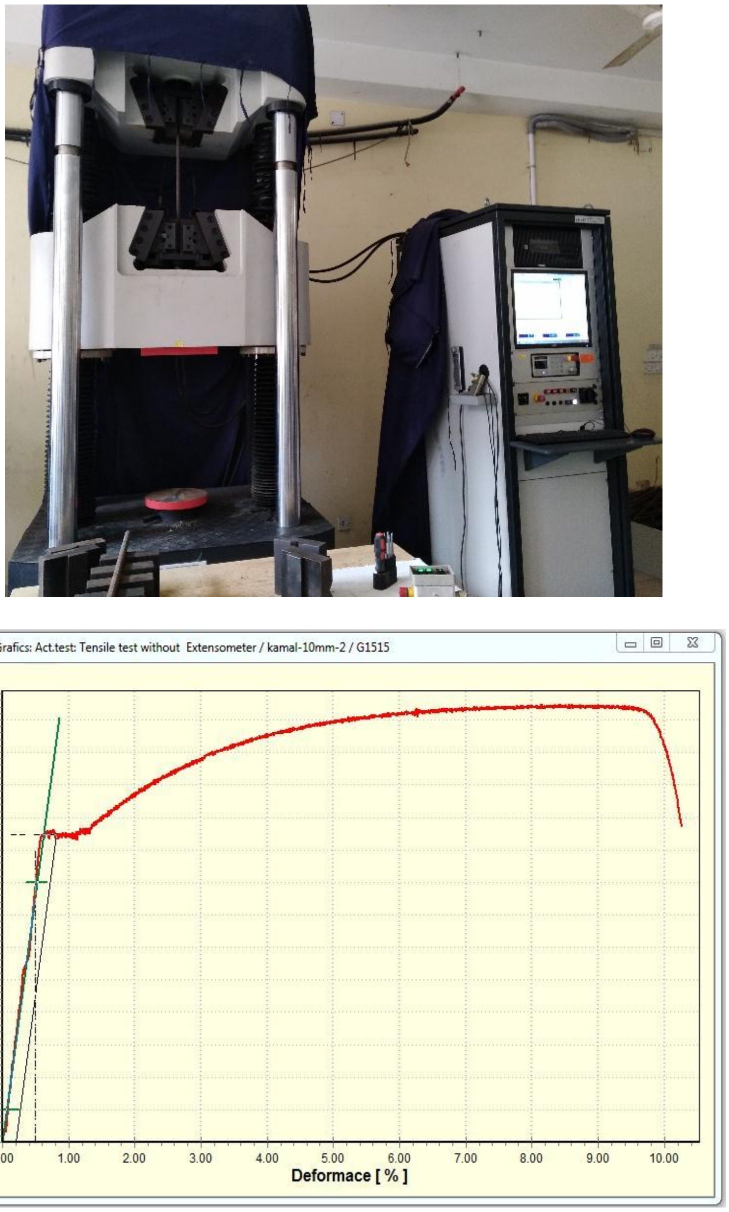


Figure 3.5: Rebar Test and Stress-Strain Curve of Rebar

Table 3.3: Mechanical Properties of Reinforcing Steel

Sl. No.	Diameter (mm)	Yield Strength (MPa)	Ultimate Strength (MPa)	Elongation (%)
1	16	494	688	15.6
2	10	463	649	10.1

3.3 MIXTURE PROPORTIONS

To investigate shear strength of RC beams, a total of 32 (16 cases × 2 specimens/case) RC beams were made. Mixture proportions of concrete are summarized in **Table 3.4.** The investigated cases are tabulated in **Table 3.5.** W/C ratio was kept at 0.50 for all cases. Air content in concrete was about 2% and it was confirmed by test **Figure 3.6**. Cement contents were 360 and 390 kg/m^3. Compressive strengths of concrete were 24 and 29 MPa.

Table 3.4: Mixture Proportions of Concrete

Concrete	Com. strength of concrete (MPa)	Unit Contents (kg/m^3)			
		Cement	Sand	Brick	Water
BA	24	360	670	809	180
	29	390	645	779	195
RBA	24	360	670	809	180
	29	390	645	779	195

BA-Virgin Brick Aggregate, RBA – Recycled Brick Aggregate

Table 3.5: Cases Investigated for Virgin Brick Aggregate (BA) and Recycled Brick Aggregate (RBA)

Notation	Com. strength of Concrete (MPa)	Steel Ratio (%)	Shear span to depth ratio (a_s/d)	Notation	Com. strength of concrete (MPa)	Steel Ratio (%)	Shear span to depth ratio (a_s/d)
BA1-0.82-24-2.04*	23.68	0.82	2.04	RBA1-0.82-24-2.04	24.12	0.82	2.04
BA2-0.82-24-2.04	23.68	0.82	2.04	RBA2-0.82-24-2.04	24.12	0.82	2.04
BA3-0.82-24-2.45	23.68	0.82	2.45	RBA3-0.82-24-2.45	24.12	0.82	2.45
BA4-0.82-24-2.45	23.68	0.82	2.45	RBA4-0.82-24-2.45	24.12	0.82	2.45
BA5-0.82-29-2.04	28.71	0.82	2.04	RBA5-0.82-29-2.04	27.5	0.82	2.04
BA6-0.82-29-2.04	28.71	0.82	2.04	RBA6-0.82-29-2.04	27.5	0.82	2.04
BA7-0.82-29-2.45	28.71	0.82	2.45	RBA7-0.82-29-2.45	27.5	0.82	2.45
BA8-0.82-29-2.45	28.71	0.82	2.45	RBA8-0.82-29-2.45	27.5	0.82	2.45
BA9-1.23-24-2.04	23.68	1.23	2.04	RBA9-1.23-24-2.04	24.12	1.23	2.04
BA10-1.23-24-2.04	23.68	1.23	2.04	RBA10-1.23-24-2.04	24.12	1.23	2.04
BA11-1.23-24-2.45	23.68	1.23	2.45	RBA11-1.23-24-2.45	24.12	1.23	2.45
BA12-1.23-24-2.45	23.68	1.23	2.45	RBA12-1.23-24-2.45	24.12	1.23	2.45
BA13-1.23-29-2.04	28.71	1.23	2.04	RBA13-1.23-29-2.04	27.5	1.23	2.04
BA14-1.23-29-2.04	28.71	1.23	2.04	RBA14-1.23-29-2.04	27.5	1.23	2.04
BA15-1.23-29-2.45	28.71	1.23	2.45	RBA15-1.23-29-2.45	27.5	1.23	2.45
BA16-1.23-29-2.45	28.71	1.23	2.45	RBA16-1.23-29-2.45	27.5	1.23	2.45

*BA1 indicates beam 1 made with brick aggregate, 0.82 indicates steel ratio, 24 indicates compressive strength of concrete in MPa, 2.04 indicates shear span to depth ratio.

The mix proportion used in this study was done on a weight basis and the unit contents of the ingredients of concrete were assumed to sum up to 1 m^3 of concrete and can be correlated by the following equation:

$$\frac{C}{G_C\gamma_w}+\frac{S}{G_S\gamma_w}+\frac{A}{G_A\gamma_w}+\frac{W}{G_w\gamma_w}+\frac{Air(\%)}{100}=1$$

Where,

C = Unit content of cement (kg/m^3 of concrete)

S = Unit content of fine aggregate (kg/m^3 of concrete)

A = Unit content of coarse aggregate (kg/m^3 of concrete)

W = Unit content of water (kg/m^3 of concrete)

γ_w = Unit weight of water ((kg/m^3)

G_c= Specific gravity of cement

G_s = Specific gravity of fine aggregate (SSD)

G_A = Specific gravity of coarse aggregate (SSD)

G_w = Specific gravity of water

Air (%) = Percentage of air in concrete (assumed at 2% without air entraining agent)

Figure 3.6: Measurement of air entrainment

3.4 DETAILS OF TEST BEAMS

Two different lengths were used (2400mm and 2100mm). All the beams were in same cross section of 200mm x 300mm. The beam designation included a combination of letters and numbers. Letters stand for whether recycled brick aggregate or virgin brick aggregate. Second two-digit indicates reinforcement percentage and 3rd two-digit indicates compressive strength of concrete. Last digits stand for shear span to depth ratio. For instance, “BA1-0.82-24-2.04” indicates beam 1 made with brick aggregate, 0.82 indicates steel ratio, 24 indicates compressive strength of concrete in MPa, 2.04 indicates shear span to depth ratio.

The longitudinal reinforcement was selected to ensure a shear failure prior to a flexural failure, yet steel remain below the maximum amount allowed by code. All of the specimens had #3 stirrups spaced at 5 in. within the bearing area to prevent premature failure as well as #3 stirrups spaced at 6 in. within the middle region to support the reinforcing cage and prevent any premature failure outside of the shear test regions. **Figure 3.7** depicts the shear force and bending moment diagram and reinforcement details and load pattern have been shown in **Figure 3.8** and **Figure 3.9.**

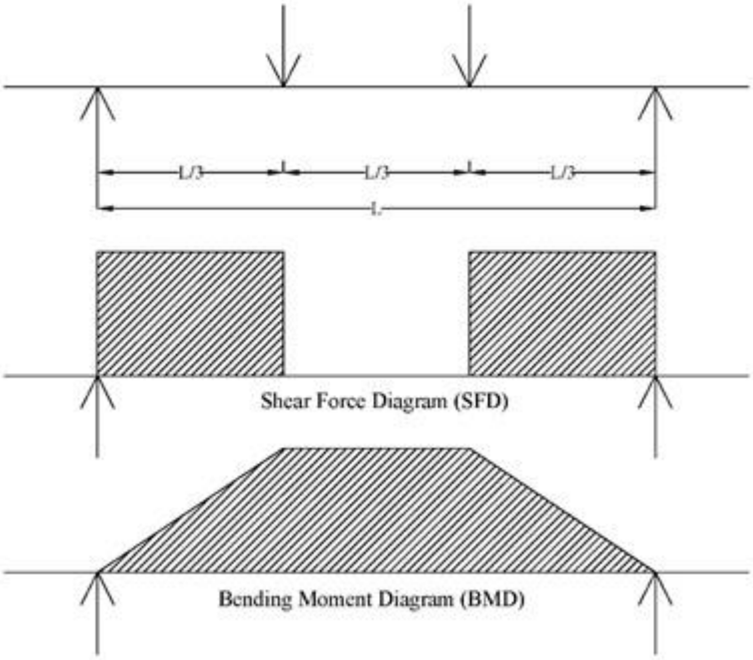


Figure 3.7: Shear force diagram and bending moment diagram

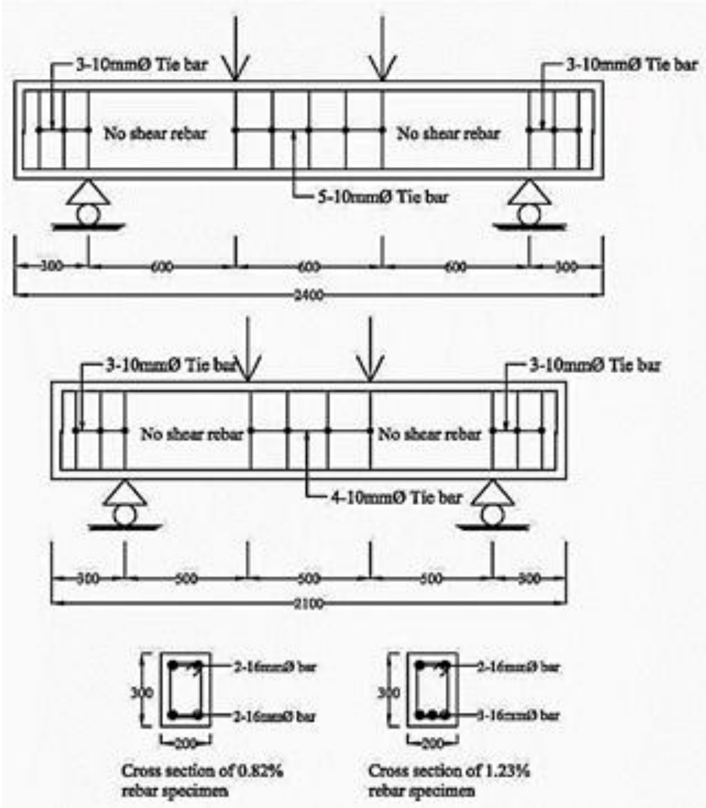


Figure 3.8: Load pattern and Reinforcement details of the test beam

Figure 3.9: Reinforcement case

3.5 BEAM FABRICATION AND CURING OF TEST SPECIMENS

All the test beams were fabricated in the Structural Engineering Laboratory at the Islamic University Technology. Steel formwork was used to cast the beams. Due to the dimension of the beams, it was possible to cast four beams at a time. Photographs show the reinforcing cages and the construction process (**Figure 3.10** and **Figure 3.11).** After casting, the top surface of the beams was covered with burlap and plastic sheeting (**Figure 3.12**). Cylinders were cured in the same environment as the test beams by placing them next to the beams. The sheeting and burlap were then removed, and the beams were allowed to air cure in the lab environment.

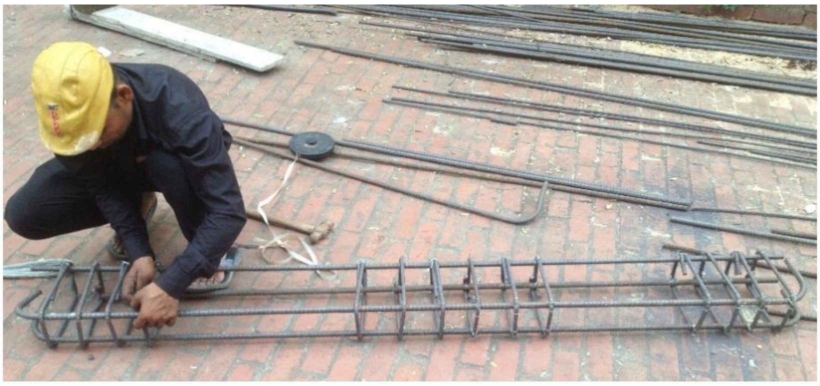
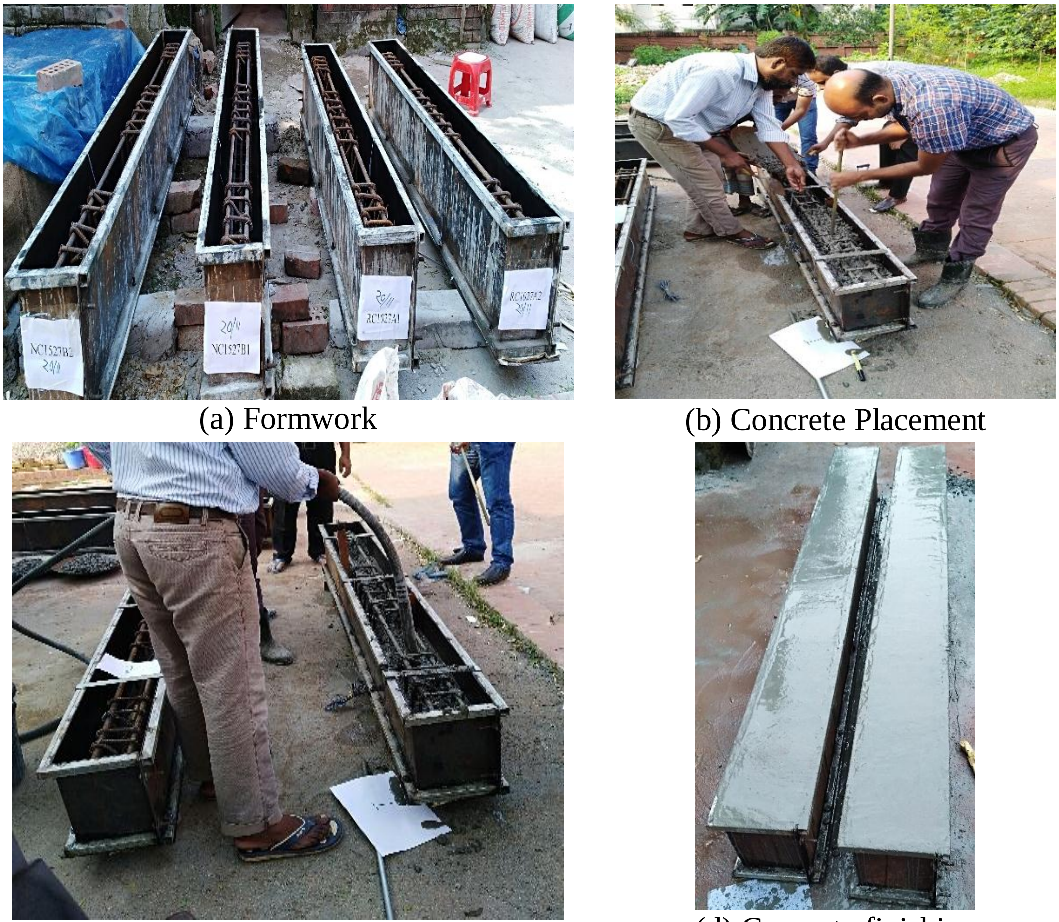


Figure 3.10: Preparing Reinforcement case



(a) Formwork

(b) Concrete Placement

(c) Concrete consolidation

(d) Concrete finishing

Figure 3.11: Beam Construction Process

Figure 3.12: Curing of casted beams

3.6 TEST SET-UP AND PROCEDURE

3.6.1 Testing facilities

A load frame was assembled and equipped intended to apply the four-point loads to the beams. The load was applied in a displacement control method with a rate of 0.50 mm/min. The shear beams were supported on a roller and pin support, 300 mm from each end of the beam.

Local Deformations and Strains.

Electrical resistance gauges were used to monitor local strains in the longitudinal steel reinforcement of the test region. They were made of constantan foil with 120-ohm resistance and had a linear pattern (uniaxial) with a gauge length of ¼ in. strain gauges were installed on longitudinal steel reinforcement in the test region**.** The strain values obtained from the strain gauges are localized measurements at the point where the gauge is installed.

Global Deformations.

Linear Variable Displacement Transducer (LVDT) was used to monitor vertical deflection of the test specimen.

3.6.2 Instrumentation

3.6.2.1 The linear variable differential t5ransducer

The specimens were instrumented with several measurement devices in order to monitor global and local deformations and strains. Linear variable differential transducer (LVDT) (**Figure 3.13)** is a type of electrical transducer used for measuring linear displacement (position). Three LVDT were used to measure displacement. Two LVDT were attached in mid share span and one LVDT was attached in the middle point of the beam.

The load was directly measured from the load cell of the actuators. All devices were connected to a data acquisition system capable of reading up to 120 channels and all the data was recorded as shown in **Figure 3.16.**

Figure 3.13: Linear variable differential transducer (LVDT)

3.6.2.2 Strain gauge

A strain gauge is a sensor whose resistance varies with applied force; it converts force, pressure, tension, weight, etc., into a change in electrical resistance which can then be measured. When external forces are applied to a stationary object, stress and strain are the result. Strain gauges were used to measure the deflection at the shear span and strain in the reinforcement. The strain gauges were installed on the lower layer of the bottom longitudinal reinforcement at mid shear span (maximum shear location). **Figure 3.14** shows the fastening procedure and the location of the strain gauges. During the test, both the deformation and strains were monitored until the beam reached failure.

Figure 3.14: Installation of the strain gauge and datalogger

3.6.3 Procedure

All the specimens were tested as simply supported and subjected to four-point loading. Universal Testing Machine (UTM) (**Figure 3.15**) was used to apply load to the beam specimens. Therefore, the test set-up required the simultaneous action as shown in **Figure 3.16** and **Figure 3.17.**

Figure 3.15: Universal Testing Machine (UTM)

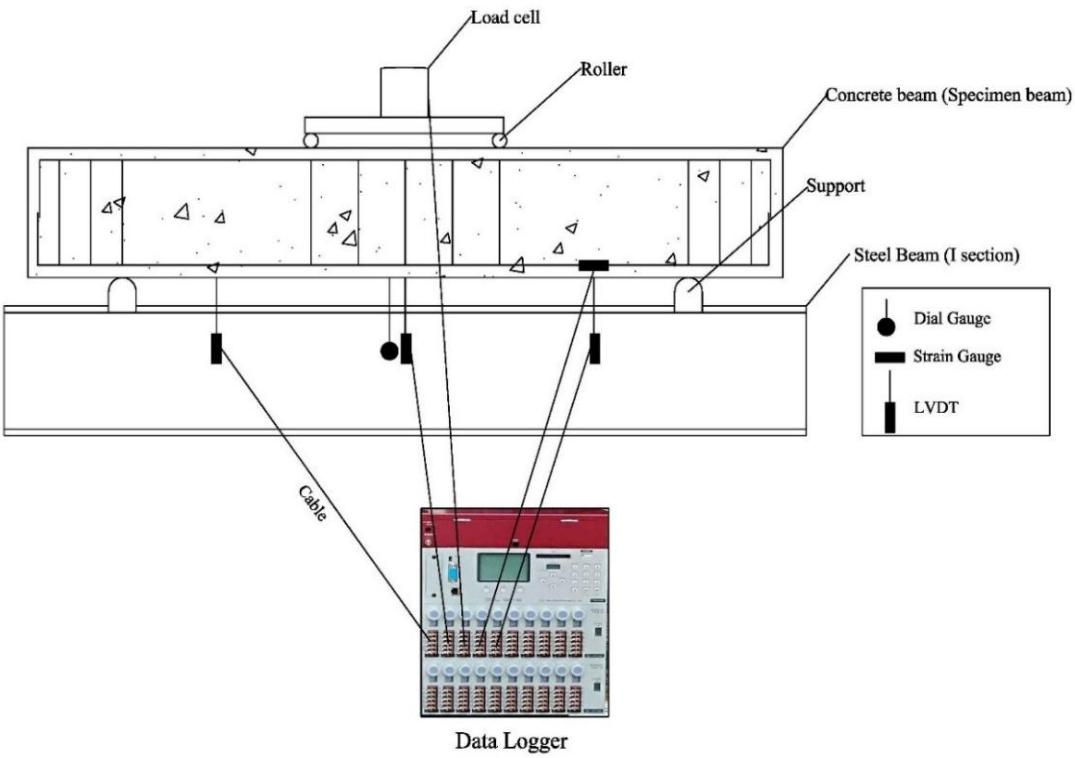


Figure 3.16: Test Setup of Beams

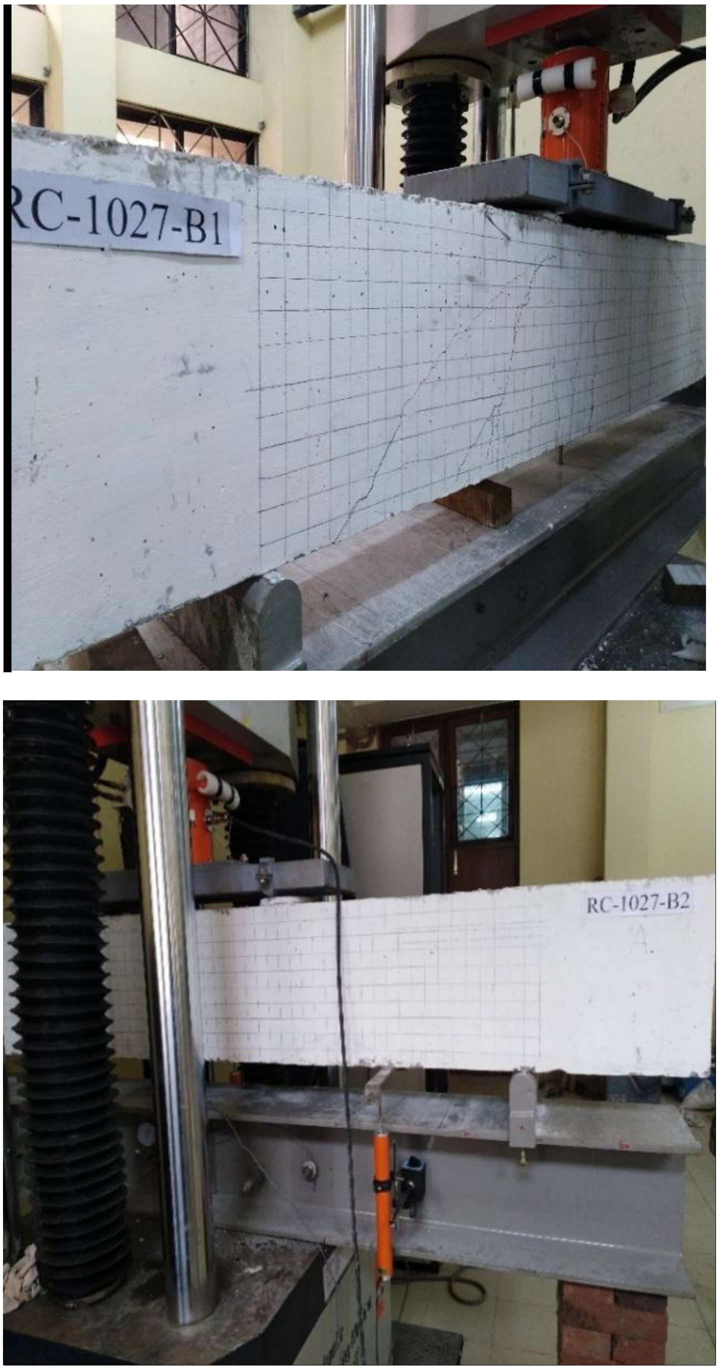


Figure 3.17: Details of Set-up

CHAPTER 4: RESULTS AND DISCUSSIONS

4.1 GENERAL

The purpose of this study was to evaluate the shear behavior of full-scale reinforced concrete (RC) beams constructed from RBA, which has not been fully investigated in previous research studies. The objectives of this section are to: (1) discuss the overall behavior of the specimens, (2) discuss the crack morphology and progression, (3) discuss the load-deflection response, (4) evaluate the failure mechanism including reinforcement strains, (5) compare the test results with predicted capacities based on applicable design standards, (6) compare the RBA test results with the control specimen results BA, and (7) compare the test results with a shear test database of conventional concrete specimens.

4.2 MATERIAL PROPERTY TEST RESULTS AND COMPARISON WITH SHEAR BEHAVIOR

Previous research and reports (ASCE-ACI426, 1973; ACI318, 2014) showed that splitting tensile strength, flexural strength, and fracture energy are important parameters affecting shear strength of concrete. For this reason, the following section compares the relationship between these parameters and shear strengths for the three mixes studied in this project. To compare the shear strengths of the BA and RBA beams, the test results must be adjusted to reflect the different compressive strengths. ACI 318 (2014) provisions use the square root of the compressive strength of concrete to determine the splitting tensile strength (**4.1)**, flexural strength (**4.2**), and shear strength (**4.3**) of a beam. In terms of fracture energy, Bazant's equation (**4.4**) uses a 0.46 power of the compressive strength of concrete to calculate the fracture energy of concrete.

$$f'_{ct} = 6.7\sqrt{f'_c} \qquad \textbf{4.1}$$

f'_c compressive strength of concrete in psi

$$fr = 7.5\sqrt{f'_c} \qquad \textbf{4.2}$$

f'_c compressive strength of concrete in psi

$$V_c = 2\sqrt{f'_c} \quad \textbf{4.3}$$

f'_c compressive strength of concrete in psi

$$G_F = 2.5\alpha_o\left(\frac{f'_c}{0.051}\right)^{0.46}\left(1+\frac{d_a}{11.27}\right)^{0.22}\left(\frac{w}{c}\right)^{-0.30} \quad \textbf{4.4}$$

Where α_0 is an aggregate shape factor (α_0=1 for rounded aggregate, and α_0=1.12 for angular aggregate), $f'c$ is the compressive strength of the concrete in psi, d_a is the maximum aggregate size in inches, and w/c is the water-to-cement ratio of the concrete.

Figure 4.1 and **Figure 4.2** offer a comparison of the splitting tensile strength, flexural strength, shear strength and fracture energy, for the two different concretes tested in this study.

For BA test beams, for low compressive strength, the splitting tensile strength, flexural strength, and fracture energy slightly increased compared to the RBA. In all cases increment is less than 1%. In other words, RBA exhibited a slight increase in basic mechanical properties and a slight decrease in shear capacity. But in higher compressive strength, the splitting tensile strength, flexural strength, and fracture energy slightly decrease compared to the RBA but the changes are not much. In summary, it can be said that mechanical properties and shear capacity more or less similar for both BA and RBA. It can also be said that the formulae for evaluation of mechanical properties and shear capacity more or less similar.

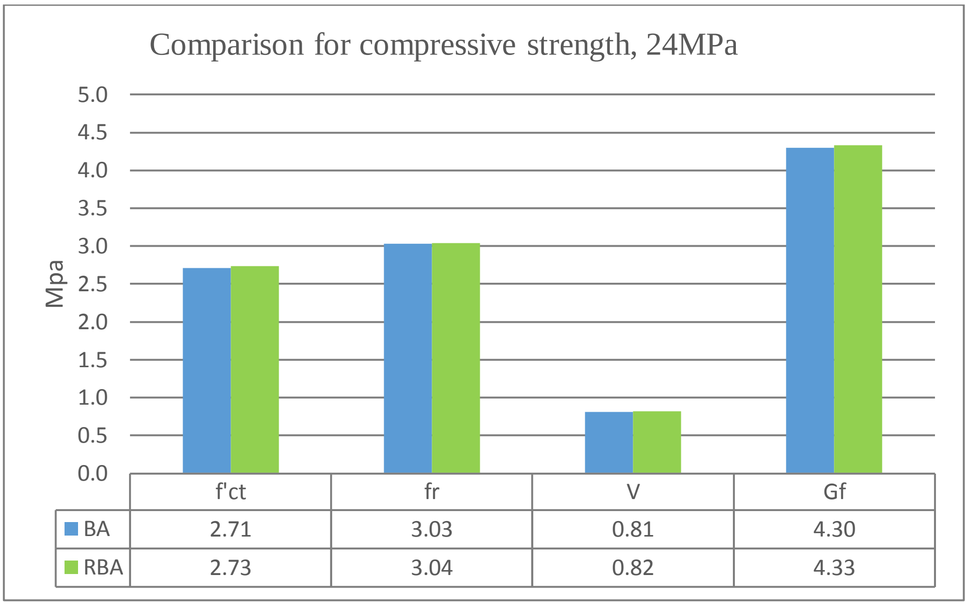


Figure 4.1: Comparison of Mechanical Properties and Shear Strengths of the BA and RBA Beams for f'c 24MPa

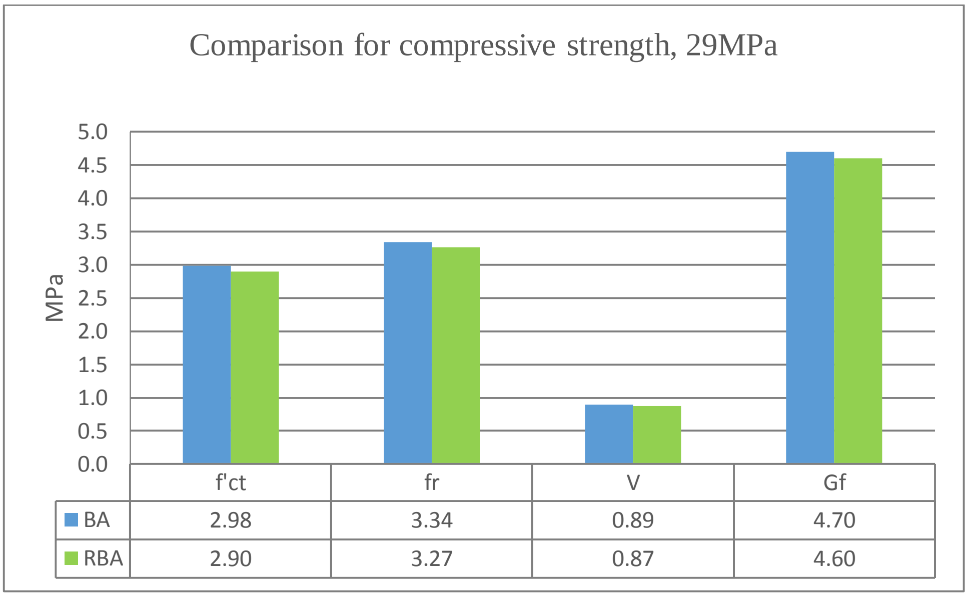


Figure 4.2: Comparison of Mechanical Properties and Shear Strengths of the BA and RBA Beams for f'c 29MPa

4.3 TEST RESULTS AND BEHAVIOR OF FULL-SCALE SPECIMENS

Table 4.1 and **Table 4.2** summarize the compressive strength (f'_c) at the time of testing, shear force at failure (V_{test}), average shear stress at failure ($V_{test}/b_w d$), and the ratio of the average shear stress to the square root of the compressive strength ($V_{test}/\sqrt{f'_c}$). A useful comparison is to compare the last column with ACI 318 (2014). From the ACI equation, the ratio of $V_{test}/\sqrt{f'_c}$ is 0.17. In **Table 4.1** and **Table 4.2**, none of the ratio ($V_{test}/\sqrt{f'_c}$) falls below BA and RBA than 0.17 which indicates that ACI equation underestimate the shear capacity. This findings also resemble with the other researcher (Arezoumandi, et al., 2014).

Table 4.1: Test Result Summary of BA concrete beam

Specimen (BA)	f'_c (Mpa)	V_{test} (kN)	$V_{test}/b_w d$ (Mpa)	$V_{test}/\sqrt{f'c}$
BA1-0.82-24-2.04	23.68	55.00	1.10	0.23
BA2-0.82-24-2.04	23.68	54.25	1.09	0.22
BA3-0.82-24-2.45	23.68	52.50	1.05	0.22
BA4-0.82-24-2.45	23.68	53.00	1.06	0.22
BA5-0.82-29-2.04	28.71	56.00	1.12	0.21
BA6-0.82-29-2.04	28.71	55.25	1.11	0.21
BA7-0.82-29-2.45	28.71	55.50	1.11	0.21
BA8-0.82-29-2.45	28.71	54.75	1.10	0.20
BA9-1.23-24-2.04	23.68	61.50	1.23	0.25
BA10-1.23-24-2.04	23.68	63.00	1.26	0.26
BA11-1.23-24-2.45	23.68	57.00	1.14	0.23
BA12-1.23-24-2.45	23.68	59.25	1.19	0.24
BA13-1.23-29-2.04	28.71	63.75	1.28	0.24
BA14-1.23-29-2.04	28.71	62.25	1.25	0.23
BA15-1.23-29-2.45	28.71	65.00	1.30	0.24
BA16-1.23-29-2.45	28.71	62.00	1.24	0.23

Table 4.2: Test Result Summary of RBA concrete beam

Specimen (RBA)	f'c (Mpa)	V_{test} (kN)	$V_{test}/b_w d$ (Mpa)	$V_{test}/\sqrt{f'c}$
RBA1-0.82-24-2.04	24.12	52.50	1.05	0.21
RBA2-0.82-24-2.04	24.12	55.50	1.11	0.23
RBA3-0.82-24-2.45	24.12	53.50	1.07	0.22
RBA4-0.82-24-2.45	24.12	57.50	1.15	0.23
RBA5-0.82-29-2.04	27.5	56.00	1.12	0.21
RBA6-0.82-29-2.04	27.5	59.50	1.19	0.23
RBA7-0.82-29-2.45	27.5	55.00	1.10	0.21
RBA8-0.82-29-2.45	27.5	56.00	1.12	0.21
RBA9-1.23-24-2.04	24.12	63.50	1.27	0.26
RBA10-1.23-24-2.04	24.12	61.00	1.22	0.25
RBA11-1.23-24-2.45	24.12	63.00	1.26	0.26
RBA12-1.23-24-2.45	24.12	59.50	1.19	0.24
RBA13-1.23-29-2.04	27.5	66.50	1.33	0.25
RBA14-1.23-29-2.04	27.5	67.50	1.35	0.26
RBA15-1.23-29-2.45	27.5	60.50	1.21	0.23
RBA16-1.23-29-2.45	27.5	63.50	1.27	0.24

4.4 LOAD-DISPLACEMENT BEHAVIOR

Mid-shear span load-displacement curves of RC beams made with different steel ratios, shear span to depth ratios, and compressive strength of concrete are shown in **Figure 4.3** to **Figure 4.5.** In both BA **(Figure 4.3)** and RBA (**Figure 4.4)** with the increase of steel ratio mid-shear span displacement of beams is reduced. A similar trend of results is also observed with the variation of compressive strength of concrete. However, with the increase of shear span to depth ratio mid-span displacement is increased. For the same load, beams made with RBA show more deflection than beams made with BA (**Figure 4.5)**.

Figure 4.3 to **Figure 4.5** shows the load-deflection behavior for the beams with different longitudinal reinforcement ratios (the deflection was measured at mid shear span). Before the first flexural cracks occurred, all of the beams displayed a steep linear elastic behavior. After additional application of load, the beams eventually developed the critical flexure-shear crack, which resulted in a drop in load. As expected, sections with a higher percentage of longitudinal reinforcement generally had a higher shear capacity, which can be attributed to a combination of additional dowel action (Taylor, 1972).

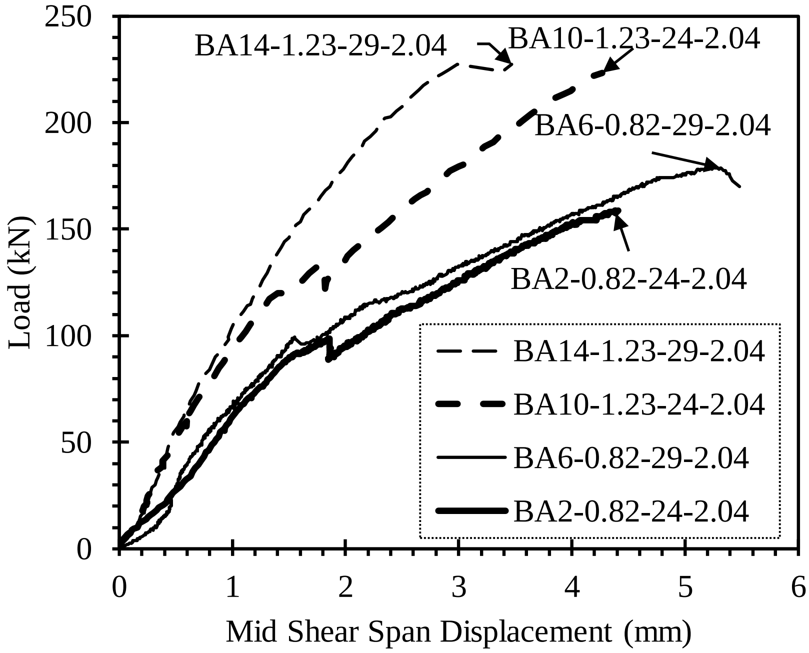


Figure 4.3: Variation with respect to Compressive Strength and Steel Ratio – BA

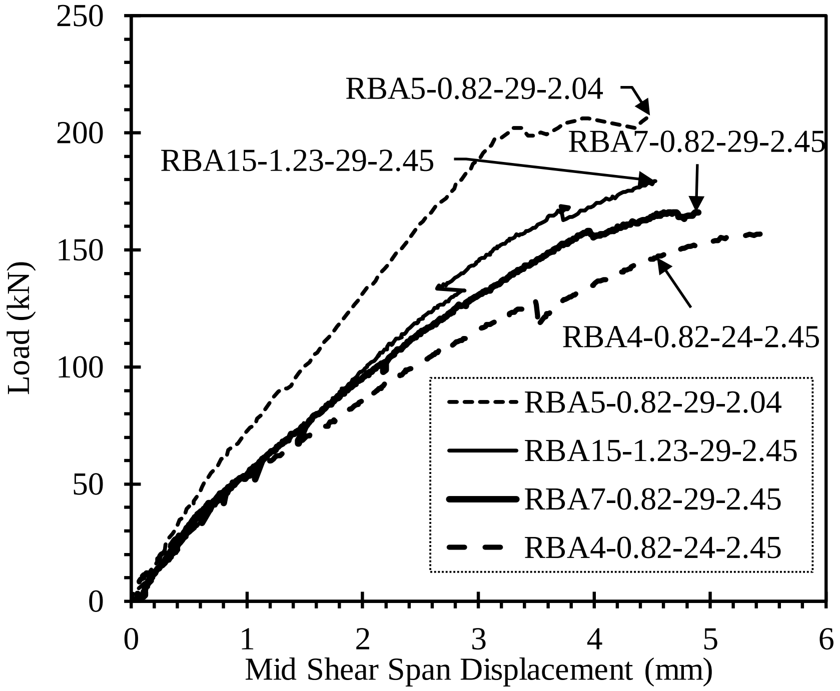


Figure 4.4: Variation with respect to Steel Ratio, Compressive Strength, and Shear Span to Depth Ratio – RBA

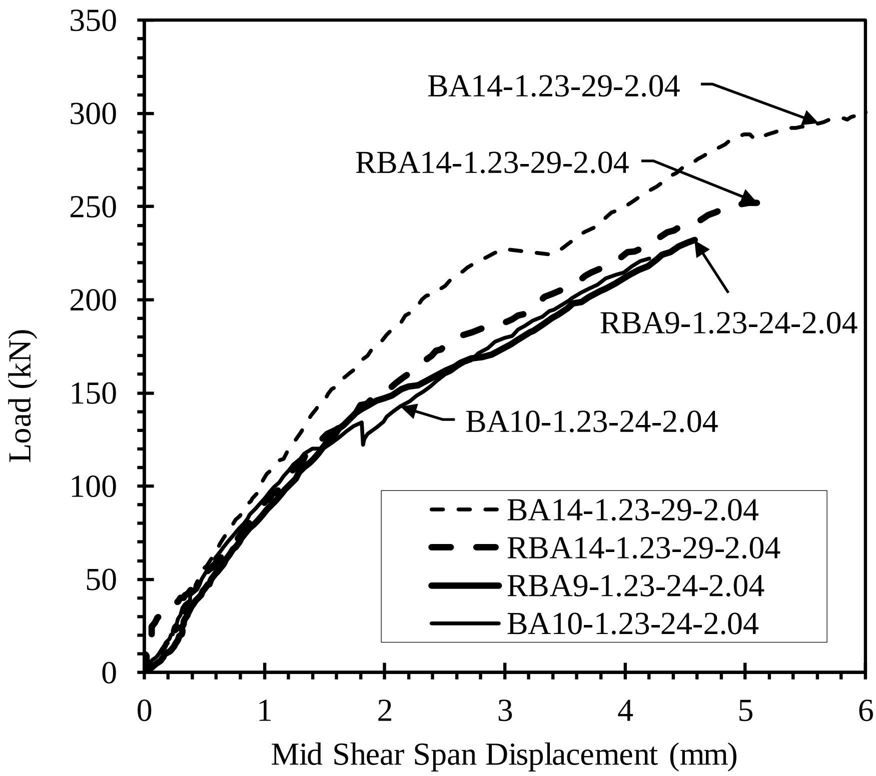


Figure 4.5: Variation with respect to Compressive Strength - RBA and BA

4.5 SHEAR CAPACITY OF RC BEAMS MADE WITH BA AND RBA

Shear strength of beams, i.e., load to form diagonal shear crack was carefully recorded by observing the initiation of diagonal shear crack on the vertical surface of the beams. The ratios ($V_{test-BA}$/ $V_{test-RBA}$) of diagonal shear cracking loads of RC beams made with BA and RBA are summarized in **Table 4.3**. The diagonal shear cracking loads for RC beam made with RBA is very similar to the corresponding RC beams made with BA. It is also seen that with the increase of compressive strength of concrete, shear capacity is increased for both RBA and BA. Similar trend of results was also found with the variation of steel ratio (ρ). However, irrespective of the types of aggregate, it is found that with an increase of shear span to depth ratio (a/d), shear capacity is reduced. This characteristics is also reported by other researcher (Raviinder, et al., 2015).

Table 4.3: Comparison of Diagonal Shear Cracking Load for BA and RBA

Specimen -BA	Load to Form Diagonal Shear Crack (kN)	Specimen – RBA	Load to Form Diagonal Shear Crack (kN)	BA/RBA
BA1-0.82-24-2.04	55.00	RBA1-0.82-24-2.04	52.50	1.05
BA2-0.82-24-2.04	54.25	RBA2-0.82-24-2.04	55.50	0.98
BA3-0.82-24-2.45	52.50	RBA3-0.82-24-2.45	53.50	0.98
BA4-0.82-24-2.45	53.00	RBA4-0.82-24-2.45	57.50	0.92
BA5-0.82-29-2.04	56.00	RBA5-0.82-29-2.04	56.00	1.00
BA6-0.82-29-2.04	55.25	RBA6-0.82-29-2.04	59.50	0.93
BA7-0.82-29-2.45	55.50	RBA7-0.82-29-2.45	55.00	1.01
BA8-0.82-29-2.45	54.75	RBA8-0.82-29-2.45	56.00	0.98
BA9-1.23-24-2.04	61.50	RBA9-1.23-24-2.04	63.50	0.97
BA10-1.23-24-2.04	63.00	RBA10-1.23-24-2.04	61.00	1.03
BA11-1.23-24-2.45	57.00	RBA11-1.23-24-2.45	63.00	0.90
BA12-1.23-24-2.45	59.25	RBA12-1.23-24-2.45	59.50	1.00
BA13-1.23-29-2.04	63.75	RBA13-1.23-29-2.04	66.50	0.96
BA14-1.23-29-2.04	62.25	RBA14-1.23-29-2.04	67.50	0.92
BA15-1.23-29-2.45	65.00	RBA15-1.23-29-2.45	60.50	1.07
BA16-1.23-29-2.45	62.00	RBA16-1.23-29-2.45	63.50	0.98
			Avg.=	0.98

V_{test} includes the weight of the steel placed on the beam for application of load, weight of load cell, and supported portion of the beam (5 KN) which were not recorded by the load cells.

4.6 PREDICTION OF SHEAR CAPACITY

Shear capacity of RC beams was calculated by using provisions of different codes and equations proposed by different researchers. The following empirical equations associated with different codes, such as ACI (2014), AASHTO (2017), CSA (2014), BS (1997), JSCE (2007), Model Code 2010 (2012), and Euro Code (2004) were used for evaluation of the shear capacity of beams made with RBA without shear reinforcement:

ACI 318M (2014)

$$V_C = 0.17\sqrt{f'_c}b_w d \tag{4.5}$$

Where, f'_c is compressive strength of concrete in Mpa, d effective depth in mm and b_w the width of the member in mm, V_c shear capacity of concrete in KN.

In complex form,

$$V_C = \left(1.9\sqrt{f'_c} + 2500\rho\frac{Vd}{M}\right) \leq 3.5\sqrt{f'_c} \tag{4.6}$$

Where, $\frac{Vd}{M} \leq 1$, f'_c is compressive strength of concrete in psi, ρ is longitudinal reinforcement ratio in percentage, d is effective depth in inch, V is total shear force in kip, M is bending moment k-in, and V_c is shear capacity of concrete in psi.

AASHTO LRFD (2017)

$$V_C = 0.0316\beta\sqrt{f'_c}b_v d_v \tag{4.7}$$

Where, β is the factor indicating the ability of diagonal cracked concrete to transmit tension, b_v is effective width of the web taken as the minimum web width within the depth in inch, d_v is effective shear depth taken as the larger value of $0.9d$ or $0.72h$ in inch, f'_c is concrete compressive strength in ksi, and V_c shear capacity of concrete in kip.

CSA Code (2014)

$$V_{cr} = \frac{245}{1275 + S_e}\sqrt{f_c'} \qquad \textbf{4.8}$$

$$S_e = \frac{35S_x}{d_{agg} + 16} \qquad \textbf{4.9}$$

Where $S_x=0.9d$, f'_c is compressive strength in MPa, d_{agg} is maximum aggregate size of concrete (mm), d is effective depth (mm)

BS code (1997)

$$v_{cr} = \frac{790}{\gamma_m}(100\rho)^{\frac{1}{3}}\left(\frac{0.4}{d}\right)^{\frac{1}{4}}\left(\frac{f_c'}{25}\right)^{\frac{1}{3}} \qquad \textbf{4.10}$$

Where, f'_c is compressive strength in MPa (f'_c<40MPa), d is the effective depth in m, γ_w is a safety factor (=1.25), $100\,\rho < 3$ and ρ is the longitudinal reinforcement ratio in percentage and v_{cr} is critical shear strength in KN.

JSCE Code (2007)

$$V_c = 0.2 \times {f'_c}^{\frac{1}{3}} \times \rho^{\frac{1}{3}} \times \left(\frac{1000}{d}\right)^{\frac{1}{4}} \times bd \qquad \textbf{4.11}$$

Where, d is the effective depth in mm, f'_c is the compressive strength of concrete in MPa, b is the width of the beam in mm, ρ is the longitudinal reinforcement ratio, b is the width of beam in mm and V_c is the shear strength in KN.

<u>Model code 2010 (2012)</u>

$$V_{Rd,c} = K_v \frac{\sqrt{f_{ck}}}{\gamma_c} b_w z \tag{4.12}$$

Where, $V_{Rd,c}$ is shear resistance in N, f_{ck} is the characteristic value of compressive strength of concrete in MPa, b_w is the width of the web in mm, z is the effective shear depth in mm, the partial safety factor γ_c=1. The parameter of the Model Code, K_V is defined by the following equation for Level I approximation:

$$K_v = \frac{180}{1000 + 1.250z} \tag{4.13}$$

<u>Euro code 2 (2004)</u>

The shear resistance of non-prestressed concrete member without shear reinforcement:

$$V_{Rk.c} = c_{Rk.c} \times k \times (100 \times \rho_l \times f_{ck})^{1/3} \times b_w \times d \tag{4.14}$$

$$k = 1 + \sqrt{200/d} \leq 2.0 \tag{4.15}$$

$$\rho_l = \frac{A_{st}}{b_w d} \leq 0.02 \tag{4.16}$$

Where, $V_{Rk.c}$ is the shear capacity in N. A_{st} is the area of the tensile reinforcement (mm^2), d is the effective depth in mm, b_w is the smallest width of the cross-section in the tensile area (mm). f_{ck} is the compressive strength of concrete in MPa, and $C_{Rk.c}$ = 0.18.

Shear capacities obtained from the experiment (V_{test}) and the provisions of codes (V_{code}) are summarized in **Table 4.4** for RBA. It is found that ACI, AASHTO, CSA, BS, Model Code 2010, JSCE, and Euro codes estimate shear capacity conservatively including extended formula of ACI. The ratio of V_{test} to V_{code} varies from 1.11 to 1.81. None of the ratio fall below 1. It is understood that the provisions of these codes can be safely used to predict the shear capacity of RC beams made with RBA. The results related to the BA aggregate are also showed similar results. These results were summarized in **Table 4.5.**

Table 4.4: V_{test}/V_{code} for Concrete Made with RBA

Specimen	V_{test}/V_{code}							
	ACI	AASHTO	CSA code	BS	Model Code	JSCE	EURO CODE	ACI – Complex
RBA1-0.82-24-2.04	1.2	1.27	1.22	1.4	1.49	1.37	1.15	1.25
RBA2-0.82-24-2.04	1.3	1.35	1.28	1.5	1.58	1.45	1.22	1.32
RBA3-0.82-24-2.45	1.3	1.30	1.24	1.5	1.52	1.39	1.18	1.29
RBA4-0.82-24-2.45	1.4	1.40	1.33	1.6	1.64	1.50	1.26	1.39
RBA5-0.82-29-2.04	1.2	1.27	1.21	1.5	1.49	1.40	1.18	1.26
RBA6-0.82-29-2.04	1.3	1.35	1.29	1.6	1.59	1.48	1.25	1.33
RBA7-0.82-29-2.45	1.2	1.25	1.19	1.4	1.47	1.37	1.16	1.25
RBA8-0.82-29-2.45	1.2	1.27	1.21	1.5	1.49	1.40	1.18	1.27
RBA9-1.23-24-2.04	1.5	1.54	1.47	1.5	1.81	1.45	1.22	1.47
RBA10-1.23-24-2.04	1.4	1.48	1.41	1.5	1.74	1.39	1.17	1.41
RBA11-1.23-24-2.45	1.5	1.53	1.46	1.5	1.79	1.44	1.21	1.48
RBA12-1.23-24-2.45	1.4	1.44	1.38	1.4	1.69	1.36	1.14	1.40
RBA13-1.23-29-2.04	1.5	1.51	1.44	1.5	1.77	1.45	1.22	1.45
RBA14-1.23-29-2.04	1.5	1.54	1.46	1.6	1.80	1.47	1.24	1.47
RBA15-1.23-29-2.45	1.3	1.38	1.31	1.4	1.61	1.32	1.11	1.34
RBA16-1.23-29-2.45	1.4	1.44	1.38	1.5	1.69	1.39	1.17	1.40
Ave.	1.4	1.40	1.33	1.5	1.64	1.41	1.19	1.36
COV (%)	7.4	7.48	7.48	3.5	7.48	3.55	3.53	6.07

Table 4.5: V_{test}/V_{code} for concrete made with BA

Specimen	V_{test}/V_{code}							
	ACI	AASHTO	CSA code	BS	Model Code	JSCE	EURO CODE	ACI – Complex
BA1-0.82-24-2.04	1.35	1.35	1.36	1.57	1.58	1.44	1.22	1.32
BA2-0.82-24-2.04	1.33	1.33	1.27	1.54	1.56	1.42	1.20	1.30
BA3-0.82-24-2.45	1.29	1.29	1.23	1.49	1.51	1.38	1.16	1.28
BA4-0.82-24-2.45	1.30	1.30	1.24	1.51	1.52	1.39	1.17	1.29
BA5-0.82-29-2.04	1.25	1.25	1.19	1.49	1.46	1.38	1.16	1.23
BA6-0.82-29-2.04	1.23	1.23	1.17	1.47	1.44	1.36	1.15	1.21
BA7-0.82-29-2.45	1.23	1.24	1.18	1.48	1.45	1.36	1.15	1.23
BA8-0.82-29-2.45	1.22	1.22	1.16	1.46	1.43	1.35	1.14	1.22
BA9-1.23-24-2.04	1.51	1.51	1.44	1.53	1.77	1.41	1.19	1.43
BA10-1.23-24-2.04	1.54	1.54	1.47	1.57	1.81	1.45	1.22	1.47
BA11-1.23-24-2.45	1.40	1.40	1.33	1.42	1.64	1.31	1.10	1.35
BA12-1.23-24-2.45	1.45	1.45	1.38	1.48	1.70	1.36	1.15	1.40
BA13-1.23-29-2.04	1.42	1.42	1.35	1.49	1.66	1.37	1.16	1.36
BA14-1.23-29-2.04	1.38	1.39	1.32	1.46	1.62	1.34	1.13	1.33
BA15-1.23-29-2.45	1.45	1.45	1.38	1.52	1.69	1.40	1.18	1.41
BA16-1.23-29-2.45	1.38	1.38	1.32	1.45	1.62	1.33	1.12	1.34
Ave.	1.36	1.36	1.30	1.50	1.59	1.38	1.16	1.32
COV (%)	7.44	7.44	7.52	2.81	7.44	2.81	2.83	5.93

Shear capacity of the RC beams was also calculated using the following equations formulated based on the fracture mechanics approach:

<u>Bazant and Yu (2005)</u>

$$V_c = 10\rho^{\frac{3}{8}}\left(1+\frac{d}{a_s}\right)\sqrt{\frac{f'_c}{1+\frac{d}{f'_c{}^{-\frac{2}{3}}3800\sqrt{d_a}}}}\,b_w d \qquad \textbf{4.17}$$

Where ρ is the longitudinal reinforcement ratio in percentage, d is the effective depth in inch, a_s is the shear span in inch, f'_c is the compressive strength of concrete psi, d_a is the maximum aggregate size in inch, b_w width of the beam in inch, and V_c shear strength of beam in pound.

Gastebled and May (2001)

$$V_c = \frac{1.018}{\sqrt{d}}\left(\frac{d}{a_s}\right)^{\frac{1}{3}} \rho^{\frac{1}{6}}\left(1-\sqrt{\rho}\right) f'_c{}^{0.35}\sqrt{E_s} b_w d \qquad \textbf{4.18}$$

Where d is the effective depth in mm, a_s is the shear span in mm, ρ is the longitudinal reinforcement ration in percentage, f'_c is the compressive strength of concrete in MPa, b_w is the width of beam in mm, E_s is the modulus of elasticity Gpa and V_c is the shear strength of concrete in N.

Xu et al. (2012)

$$V_c = \frac{1.018}{\sqrt{d}}\left(\frac{d}{a_s}\right)^{\frac{1}{3}} \rho^{\frac{1}{6}}(1-\sqrt{\rho})^{\frac{2}{3}}(0.0255 f'_c + 1.024)\, b_w d \qquad \textbf{4.19}$$

Where, d is the effective width in m, as is the span in m, ρ is the longitudinal reinforcement ration in percentage, f'_c is the compressive strength of concrete in MPa, b_w is the width of the beam in mm and V_c is the shear strength of concrete in KN.

Zsutty equation (1968)

$$V_c = 2210\left(f'_c \rho \frac{d}{a_s}\right)^{\frac{1}{3}} b_w d \qquad \textbf{4.20}$$

Where f'_c is the compressive strength of concrete in MPa, d is the effective width in mm, b_w is the width of the beam in mm and V_c is the shear strength of concrete in KN.

<u>Niwa et al. (1986)</u>

$$V_c = 0.2 \times f'_c{}^{\frac{1}{3}} \times (100\rho)^{\frac{1}{3}} \times \left(\frac{1000}{d}\right)^{\frac{1}{4}} \times \left(0.75 + 1.4\frac{a}{d}\right) \quad \textbf{4.21}$$

Where, d is the effective depth in mm, f'_c is the compressive strength of concrete in MPa, ρ is the longitudinal reinforcement ratio, a is the shear span, b is the width of the beam and V_c is the shear strength in MPa.

Table 4.6: $V_{test}/V_{Fracture\ Mechanics}$ for concrete beam made with RBA

	$V_{test}/V_{Fracture\ Mechanics}$				
Specimen	**Bazant et al.**	**Gasteble d et al.**	**Xu et al.**	**Zsutty**	**Niwa et al.**
RBA1-0.82-24-2.04	1.36	1.16	0.94	1.03	1.08
RBA2-0.82-24-2.04	1.44	1.23	0.99	1.09	1.15
RBA3-0.82-24-2.45	1.46	1.26	1.02	1.12	1.11
RBA4-0.82-24-2.45	1.57	1.35	1.09	1.20	1.19
RBA5-0.82-29-2.04	1.38	1.19	0.95	1.05	1.11
RBA6-0.82-29-2.04	1.47	1.26	1.01	1.12	1.18
RBA7-0.82-29-2.45	1.44	1.24	0.99	1.10	1.09
RBA8-0.82-29-2.45	1.46	1.26	1.01	1.12	1.11
RBA9-1.23-24-2.04	1.42	1.35	1.08	1.25	1.31
RBA10-1.23-24-2.04	1.36	1.29	1.03	1.20	1.26
RBA11-1.23-24-2.45	1.49	1.42	1.14	1.32	1.30
RBA12-1.23-24-2.45	1.40	1.34	1.07	1.24	1.23
RBA13-1.23-29-2.04	1.41	1.35	1.07	1.25	1.32
RBA14-1.23-29-2.04	1.43	1.37	1.09	1.27	1.34
RBA15-1.23-29-2.45	1.36	1.30	1.04	1.21	1.20
RBA16-1.23-29-2.45	1.43	1.37	1.09	1.27	1.26
Ave.	1.43	1.30	1.04	1.18	1.20
COV (%)	3.87	5.54	5.30	7.38	7.37

Table 4.6 and **Table 4.7** summarizes the shear capacity of RC beams obtained from fracture mechanics theory and experiments. Equations proposed by Bazant et al. (2005) and Gastebled et al. (2001) conservatively estimates the shear capacity of beams made with RBA. However, equations proposed by Xu (2012), Z_{sutty} (1968) and Niwa et al(1986) marginally estimate the shear capacity of RC beams compared to the test results. Similar results were also observed for the beams made with BA.

Table 4.7: $V_{test}/V_{Fracture\ Mechanics}$ for concrete beam made with BA

Specimen	$V_{test}/V_{Fracture\ Mechanics}$				
	Bazant et al.	**Gastebled et al.**	**Xu et al.**	**Zsutty**	**Niwa et al.**
BA1-0.82-24-2.04	1.43	1.23	0.99	1.09	1.14
BA2-0.82-24-2.04	1.41	1.21	0.98	1.07	1.13
BA3-0.82-24-2.45	1.45	1.24	1.00	1.10	1.09
BA4-0.82-24-2.45	1.46	1.26	1.01	1.12	1.10
BA5-0.82-29-2.04	1.36	1.17	0.93	1.04	1.09
BA6-0.82-29-2.04	1.34	1.15	0.92	1.03	1.08
BA7-0.82-29-2.45	1.43	1.23	0.98	1.10	1.08
BA8-0.82-29-2.45	1.41	1.21	0.97	1.08	1.07
BA9-1.23-24-2.04	1.38	1.31	1.05	1.07	1.28
BA10-1.23-24-2.04	1.41	1.34	1.08	1.09	1.31
BA11-1.23-24-2.45	1.35	1.29	1.03	1.05	1.18
BA12-1.23-24-2.45	1.41	1.34	1.08	1.09	1.23
BA13-1.23-29-2.04	1.33	1.27	1.01	1.04	1.24
BA14-1.23-29-2.04	1.30	1.24	0.99	1.01	1.21
BA15-1.23-29-2.45	1.44	1.38	1.09	1.12	1.27
BA16-1.23-29-2.45	1.37	1.31	1.04	1.07	1.21
Ave.	1.39	1.26	1.01	1.07	1.17
COV (%)	3.25	5.10	4.94	3.01	6.97

4.7 CRACK PATTERN

4.7.1 Typical Crack Progression

In addition to studying the behavior of the specimens, the crack patterns experienced by the beams were also evaluated. During testing, cracks within the test region were marked using a permanent marker after each load step. Typical crack pattern progressions are shown in **Figure 4.6**. Furthermore, **Figure 4.7** to **Figure 4.10** shows the crack pattern for the BA and RBA beams with different percentages of longitudinal reinforcement, compressive strength and shear span to depth ratio. Cracks typically began on the tension face of the beam near the loading points. As the loading progressed, the flexural cracks in the shear test region formed inclined flexure-shear cracks. The formation of the inclined flexure-shear crack did not result in immediate failure, and an additional load was required prior to failure. In general, the failure crack typically extended from the beam support to the loading point on the top side of the beam.

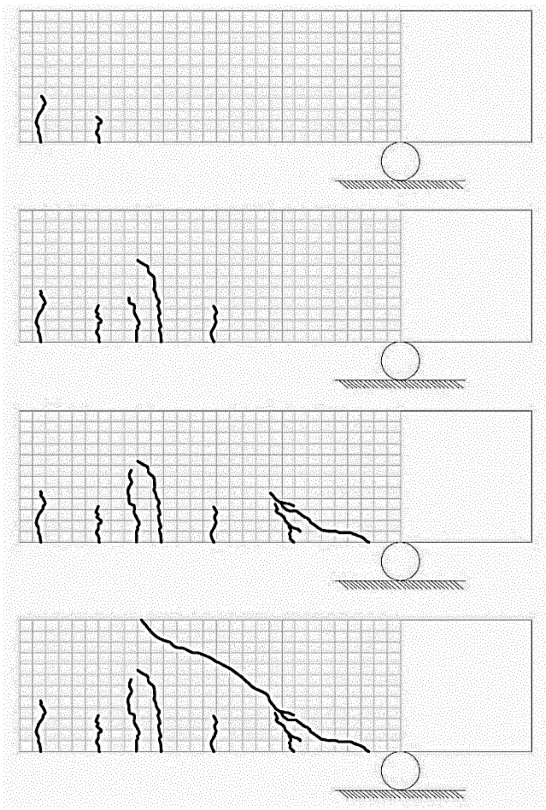


Figure 4.6: Typical crack progression

4.7.2 Crack Pattern of the Beams

The crack maps of the RC beam are shown in **Figure 4.7** to **Figure 4.10.** Based on the experimental observation, it was found that as the load is increased, the flexural cracks appear at the middle of the beam. These flexural cracks propagate vertically and it remains below the neutral axis. With further increase of load, diagonal shear cracks are formed. The load at the instant of diagonal shear crack formation was recorded carefully. Upon further increase of load, the diagonal cracks propagate to the compression face of the beam and finally causes to failure as typical shear failure. Typical shear failure of the RC beams was observed irrespective of steel ratio, shear span to depth ratio, and variation of compressive strength of concrete. Relatively more flexural cracks were observed for the beams made with more steel ratio. No significant difference in crack patterns was observed between RC beams made with BA and RBA.

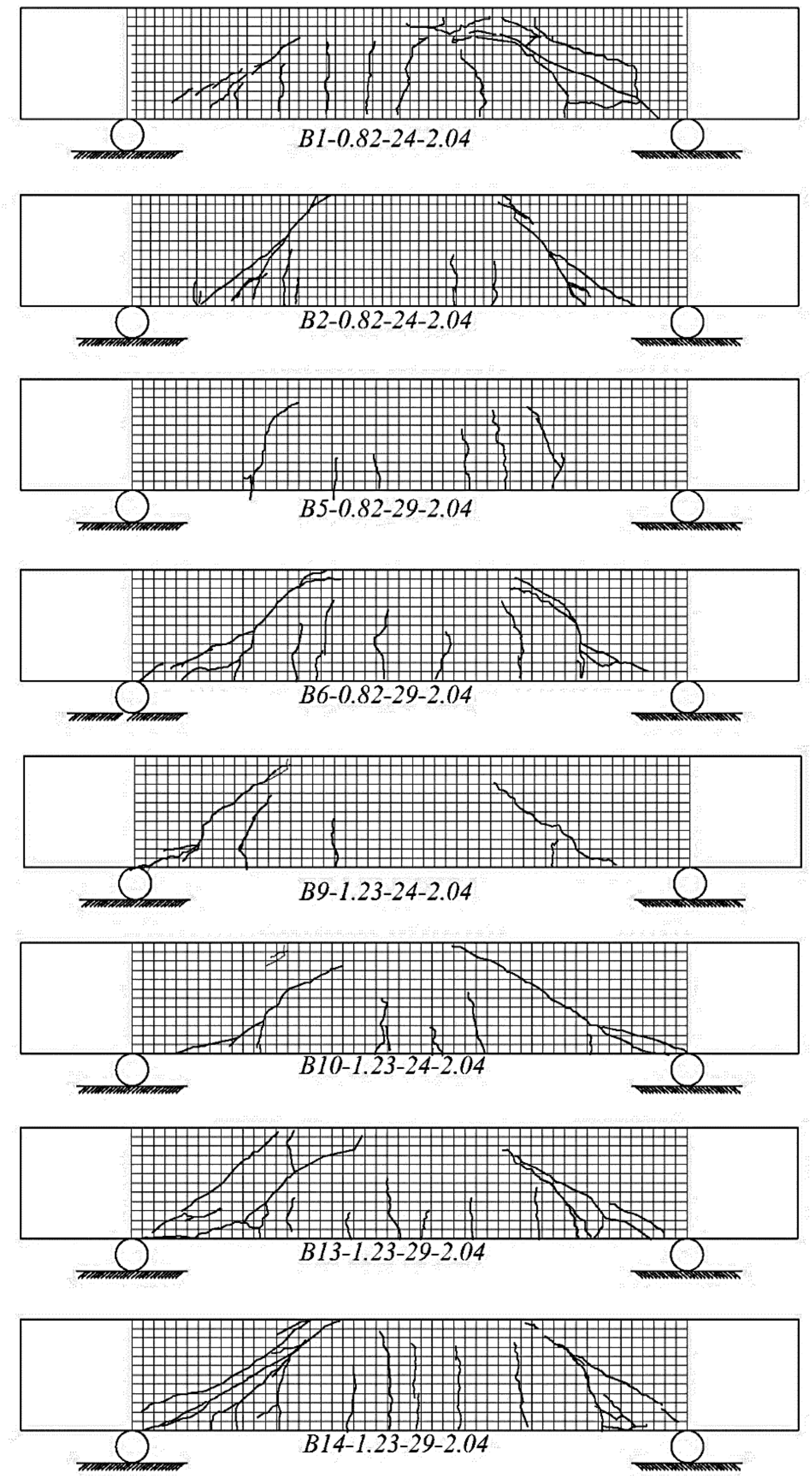


Figure 4.7: Failure Patterns of Brick Aggregate (BA) Beams

(Shear-span-to-depth ratio = 2.04)

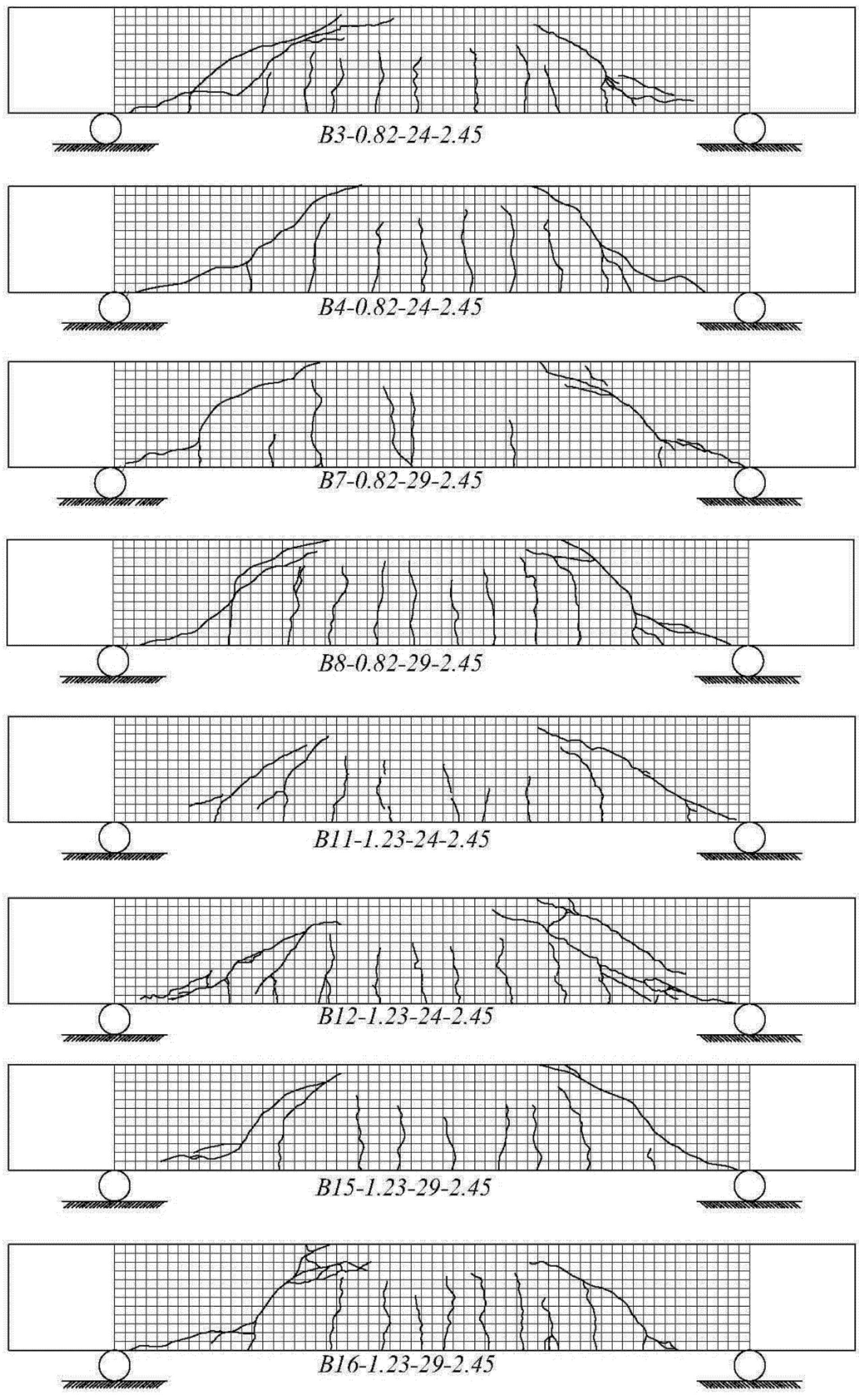


Figure 4.8: Failure Patterns of Brick Aggregate (BA) Beams

(Shear-span-to-depth ratio = 2.45)

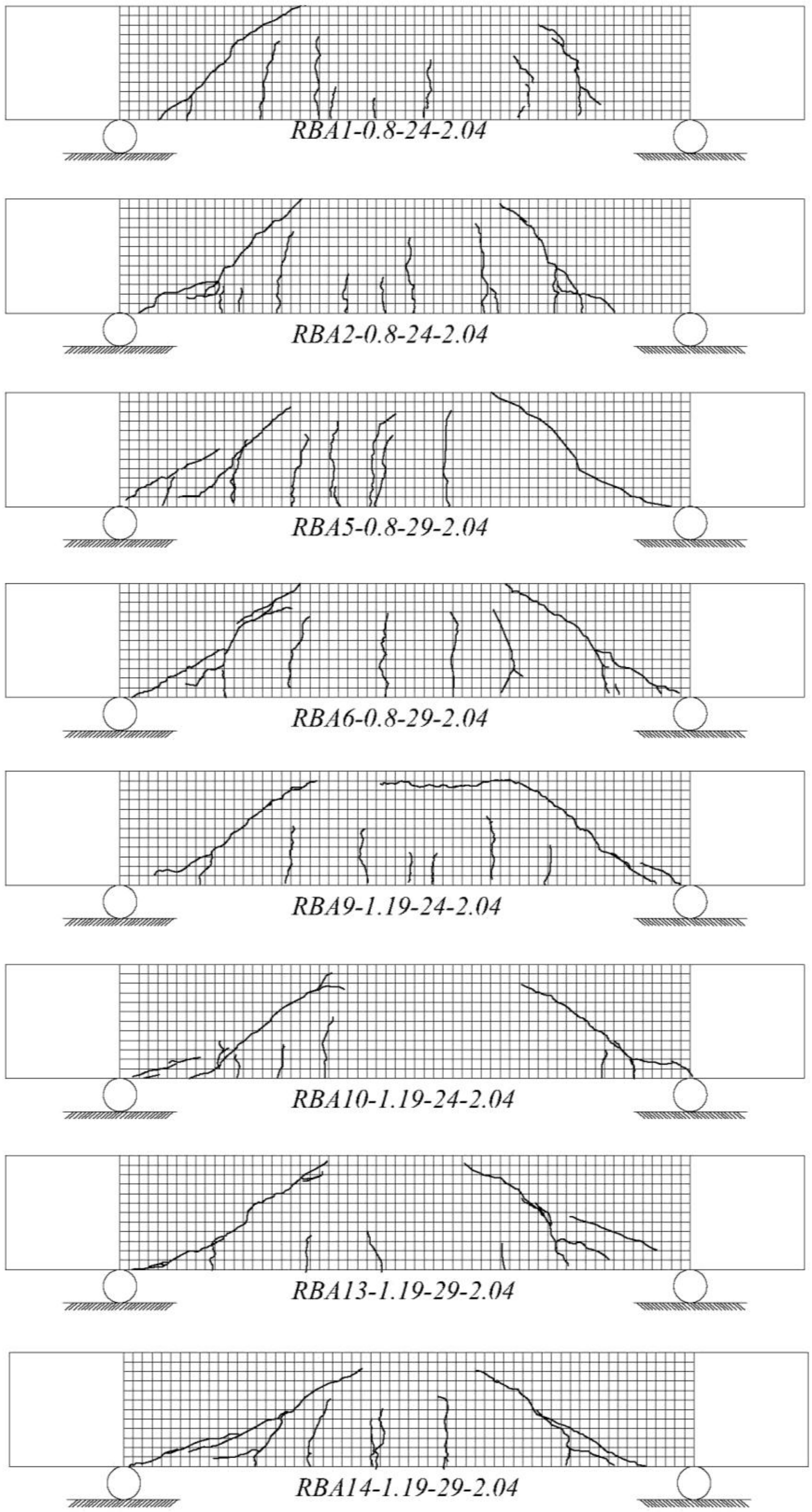


Figure 4.9: Failure Patterns of Recycled Brick Aggregate Beams

(Shear-span-to-depth ratio = 2.45)

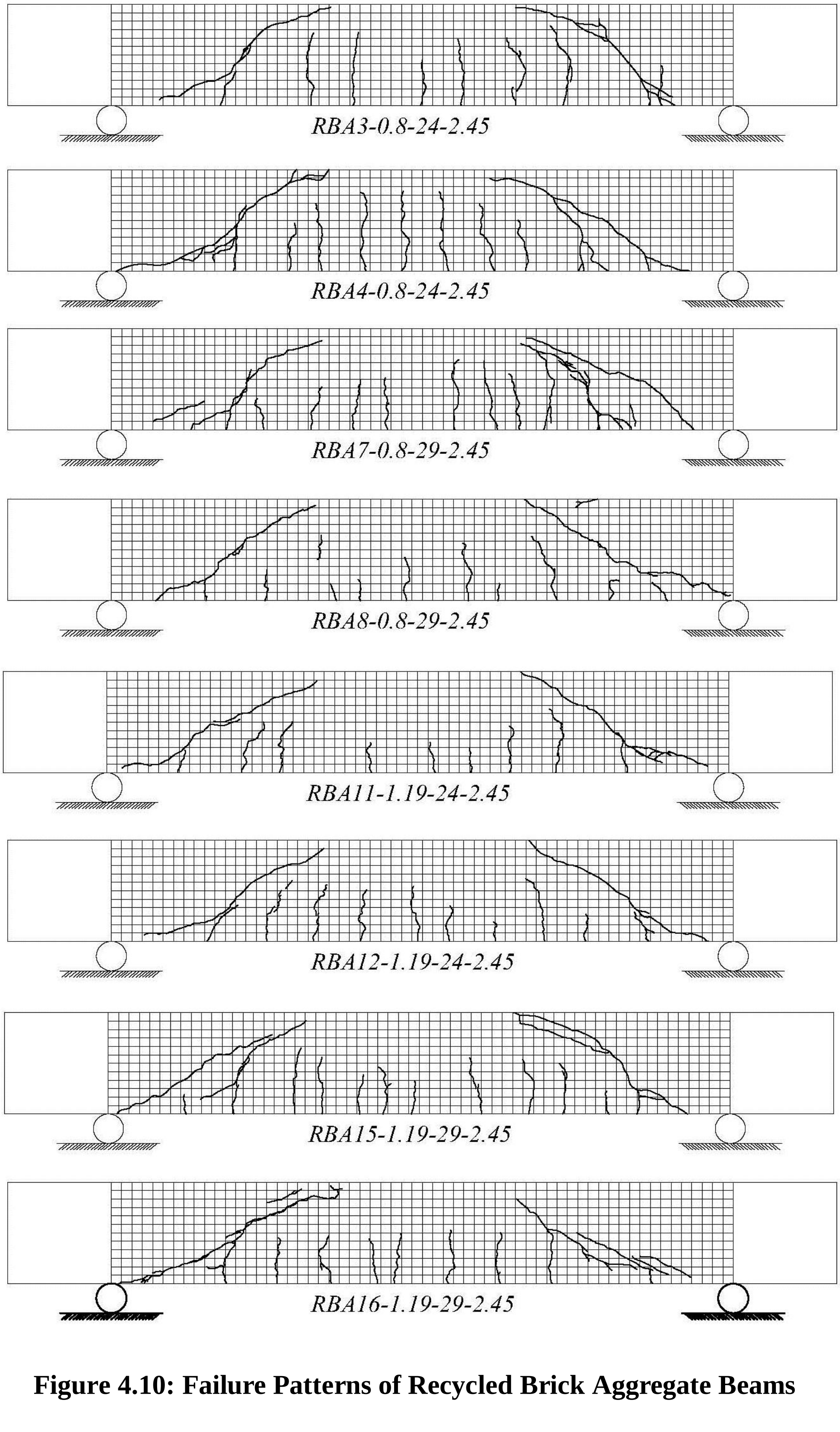


Figure 4.10: Failure Patterns of Recycled Brick Aggregate Beams

(Shear-span-to-depth ratio = 2.45)

4.8 STRAINS OVER LONGITUDINAL REINFORCEMENT

Strain over the longitudinal reinforcement was calculated by using the modified compression field theory (MCFT) method which was adopted in AASHTO LRFD (2017). As per this guideline, the following equation was used to calculate strain over the steel:

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + |V_u|\right)}{E_s A_s} \qquad \textbf{4.22}$$

Where, ε_s is the strain in non-prestressed longitudinal tension reinforcement. M_u is the moment at section in kip-inch; d_v is effective shear depth in inch, V_u is factored shear force at section in kip, E_s is modulus of elasticity of reinforcing bars in ksi, and A_s is the area of non-prestressed tension reinforcement in square inch.

Table 4.8: Comparison of Reinforcement Strain from Experiment and AASHTO LRFD (2017) Equation for RBA

Specimen	Strain Calculated, ε_s	Strain Observed, ε_s	Strain Calc./Obs.
RBA1-0.82-24-2.04	1350	586	2.30
RBA2-0.82-24-2.04	1523	1150	1.32
RBA3-0.82-24-2.45	1523	871	1.75
RBA4-0.82-24-2.45	1641	864	1.90
RBA5-0.82-29-2.04	1443	886	1.63
RBA6-0.82-29-2.04	1537	1122	1.37
RBA7-0.82-29-2.45	1420	809	1.76
RBA8-0.82-29-2.45	1597	1248	1.28
RBA9-1.23-24-2.04	1220	760	1.60
RBA10-1.23-24-2.04	812	780	1.04
RBA11-1.23-24-2.45	1202	742	1.62
RBA12-1.23-24-2.45	1133	683	1.66
RBA13-1.23-29-2.04	1149	579	1.98
RBA14-1.23-29-2.04	1344	784	1.71
RBA15-1.23-29-2.45	1123	677	1.66
RBA16-1.23-29-2.45	1251	1016	1.23
		Avg.=	1.61

The experimental (obtained from strain gauges fastened over longitudinal steel at the middle of shear span) and calculated results are summarized in **Table 4.8** and **Table 4.9.** It is found that AASHTO LRFD (2017) equation underestimates strain over the longitudinal reinforcement of RC beams made with RBA. A similar observation was also found for RC beams made with BA.

Table 4.9: Comparison of Reinforcement Strain from Experiment and AASHTO LRFD (2017) Equation for BA

Beam Notation	Strain Calculated, ε_s	Strain Observed, ε_s	Strain Calc./Obs.
BA1-0.82-24-2.04	1417	580	2.44
BA2-0.82-24-2.04	1397	610	2.29
BA3-0.82-24-2.45	1494	920	1.62
BA4-0.82-24-2.45	1509	1383	1.09
BA5-0.82-29-2.04	1443	770	1.87
BA6-0.82-29-2.04	1423	730	1.95
BA7-0.82-29-2.45	1582	1374	1.15
BA8-0.82-29-2.45	1560	1086	1.44
BA9-1.23-24-2.04	1060	490	2.16
BA10-1.23-24-2.04	1086	703	1.55
BA11-1.23-24-2.45	1084	721	1.50
BA12-1.23-24-2.45	1128	663	1.70
BA13-1.23-29-2.04	1100	415	2.65
BA14-1.23-29-2.04	1073	436	2.46
BA15-1.23-29-2.45	1241	921	1.35
BA16-1.23-29-2.45	1182	1053	1.12
		Avg.=	1.77

The variations of strain over the steel bars for some cases of BA are shown in **Figure 4.11.** Before the formation of diagonal crack all the specimen show similar strain. But with further increase of load after formation of diagonal crack, with the increase of longitudinal reinforcement (BA1-0.82-24-2.04, BA9-1.23-24-2.04), strain over the steel is reduced. A similar trend of the result is also observed for compressive strength (BA13-1.23-29-2.04, BA9-1.23-24-2.04). However, with an increase of shear span, the strain over steel is increased (BA9-1.23-24-2.04, BA12-1.23-24-2.45). Similar characteristics are found for RBA. In **Figure 4.12** , the strain of RBA and BA is compared, it is found that RBA shows more strain than BA though before initiation of shear crack no significant difference is found for both BA and RBA.

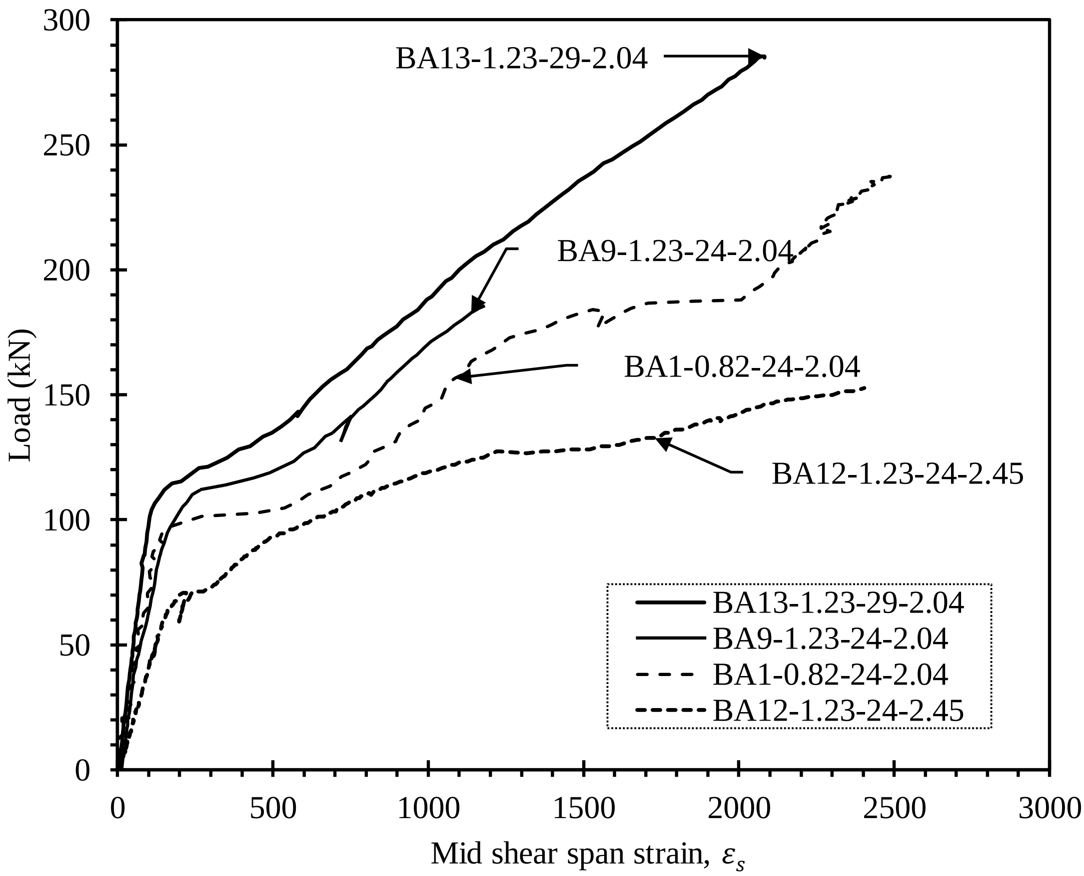


Figure 4.11: Mid shear span strain comparison of BA

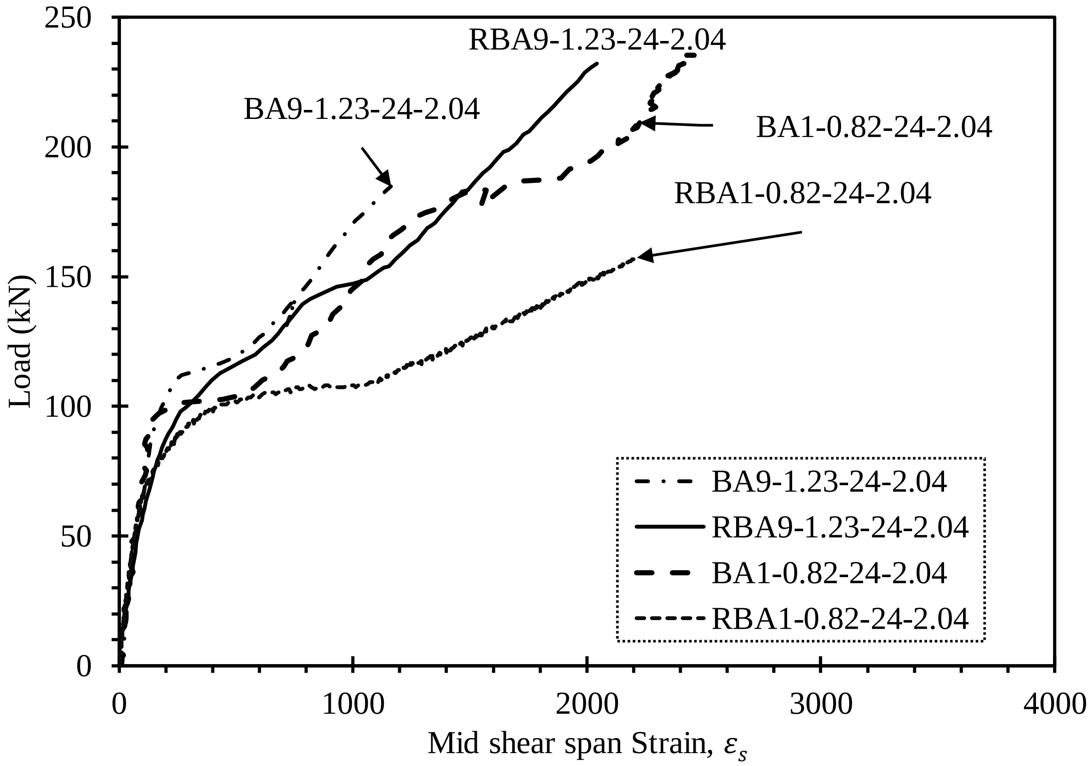


Figure 4.12: Mid shear span strain comparison of BA and RBA

4.9 STATISTICAL DATA ANALYSIS

Statistical tests were used to evaluate whether there is any statistically significant difference between the normalized shear strength of the BA and the RBA beams. Both parametric and nonparametric statistical tests were performed.

4.9.1 Parametric Test

The paired t-test is a statistical technique used to compare two population means. This test assumes that the differences between pairs are normally distributed. If this assumption is violated, the paired t-test may not be the most powerful test. The hypothesis for the paired t-test is as follows:

Ho1: The means of the normalized shear capacity of the BA is equal to the RBA beams.

Ho2: The means of the normalized shear capacity of the BA is not equal to the RBA beams.

The statistical computer program SPSS 17 was employed to perform these statistical tests. Both Kolmogorov-Smirnov and Shapiro-Wilk tests (**Figure 4.13**) showed the data, the differences between the shear capacities of the BA and the RBA beams follow a normal distribution. This is also seen from the histogram and Normal Q-Q graph (**Figure 4.14 and Figure 4.15**). Therefore, the paired t-tests could be performed. The result of the paired t-test showed that the p-values were 0.994 (**Figure 4.16**) >0.05 the hypothesis. This confirms the null hypothesis at the 0.05 significance level. In other words, the means of the normalized shear capacity of the BA was equal to the RBA beams.

4.9.2 Nonparametric Test

Unlike the parametric tests, nonparametric tests are referred to as distribution-free tests. These tests have the advantage of requiring no assumption of normality, and they usually compare medians rather than means. The Wilcoxon signed-rank test is usually identified as a nonparametric alternative to the paired t-test. The hypothesis for this test is the same as those for the paired t-test. The Wilcoxon signed-rank test assumes that the distribution of the difference of pairs is symmetrical. This assumption can be checked; if the distribution is normal, it is also symmetrical. As mentioned earlier, the data follows the normal distribution and the Wilcoxon signed-rank test (**Figure 4.17**) can be used. The p-values for the Wilcoxon signed rank was 0.121 >0.05 the hypothesis. That confirmed the null hypothesis at the 0.05 significance level.

Overall, the results of the statistical data analyses showed that the BA beams had almost the same normalized shear strength as the RBA beams.

	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Statistic	df	Sig.	Statistic	df	Sig.
BA	.139	16	.200*	.973	16	.889
RBA	.112	16	.200*	.950	16	.486

a. Lilliefors Significance Correction

*. This is a lower bound of the true significance.

Figure 4.13: Test result of Kolmogorov-Smirnov and Shapiro-Wilk test

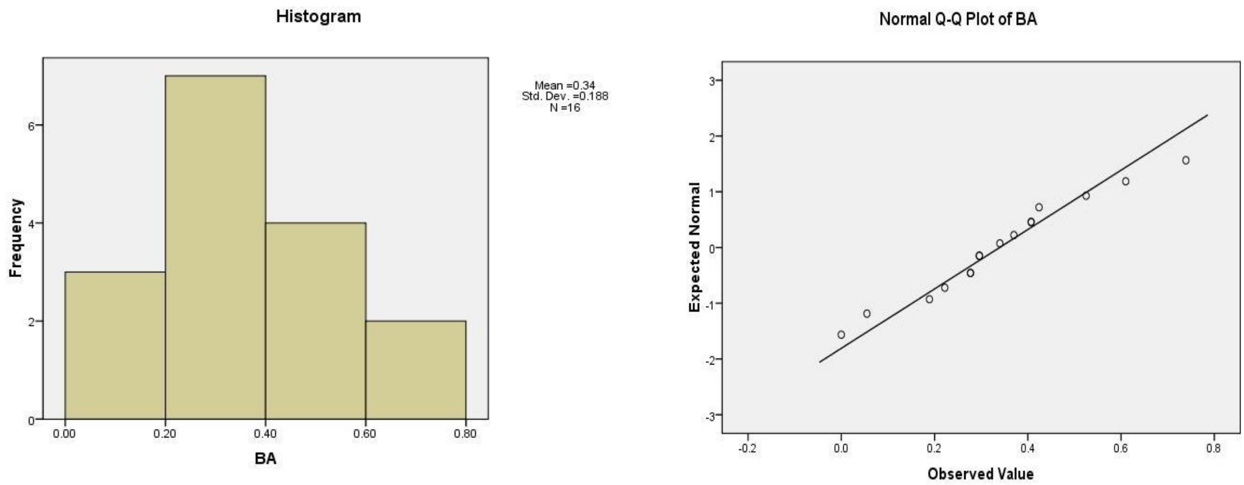


Figure 4.14: Histogram and Normal Q-Q plot of BA

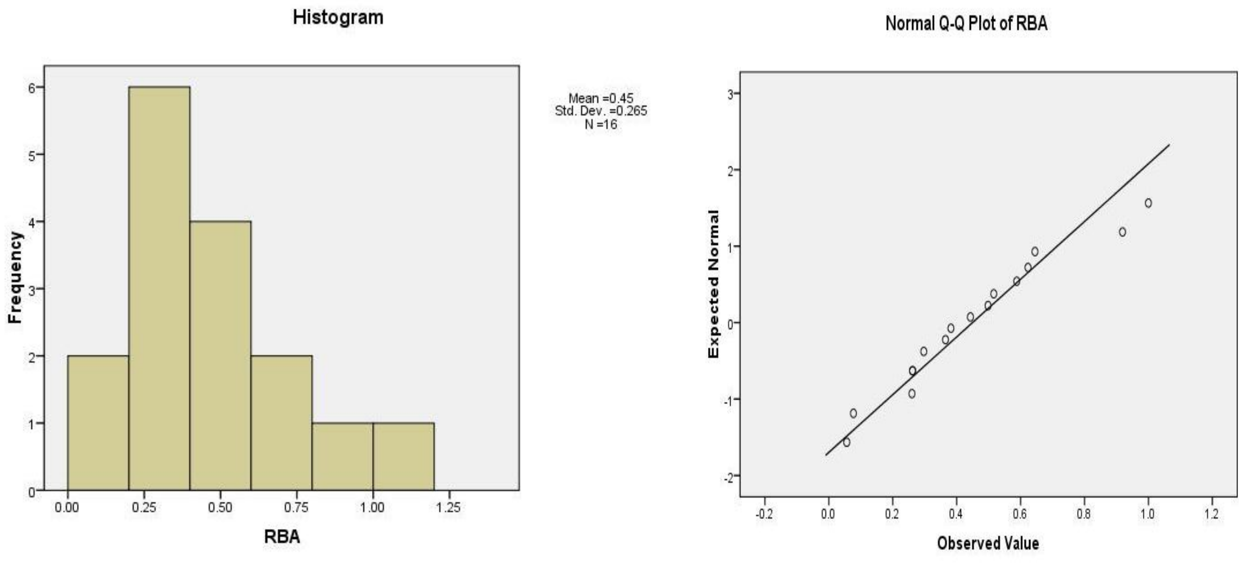


Figure 4.15: Histogram and Normal Q-Q plot of RBA

		N	Correlation	Sig.
Pair 1	BA & RBA	16	.002	.994

Figure 4.16: Paired sample correlations

Ranks

		N	Mean Rank	Sum of Ranks
RBA - BA	Negative Ranks	4[a]	9.50	38.00
	Positive Ranks	12[b]	8.17	98.00
	Ties	0[c]		
	Total	16		

a. RBA < BA

b. RBA > BA

c. RBA = BA

Test Statistics[b]

	RBA - BA
Z	-1.551[a]
Asymp. Sig. (2-tailed)	.121

a. Based on negative ranks.

b. Wilcoxon Signed Ranks Test

Figure 4.17: Test result of the Wilcoxon signed-rank test

4.10 COMPARISON OF TEST RESULTS WITH SHEAR DATABASE

The shear database (Shilang, et al., 2012) of the RC beam is an important resource for comparison of the experimental results. The four key parameters that affect concrete contribution to shear strength include depth of member or size effect (d), shear span to depth ratio (a/d), compressive strength of concrete (f´c), and longitudinal reinforcement ratio (ρ) (Reineck, et al., 2003)). To evaluate the effect of the aforementioned parameters on shear strength of the beams, the results of this study were compared with the wealth of shear test data. The test results obtained from this study with data obtained from the shear database are shown in **Figure 4.18** with the variation of compressive strength of concrete, shear span to depth ratio, effective depth, and steel ratio. The round shaped symbols of the graphs indicate the data obtained from the shear database and the triangularly shaped marks indicate the data obtained from the experiment BA while rectangular shape depics the RBA.

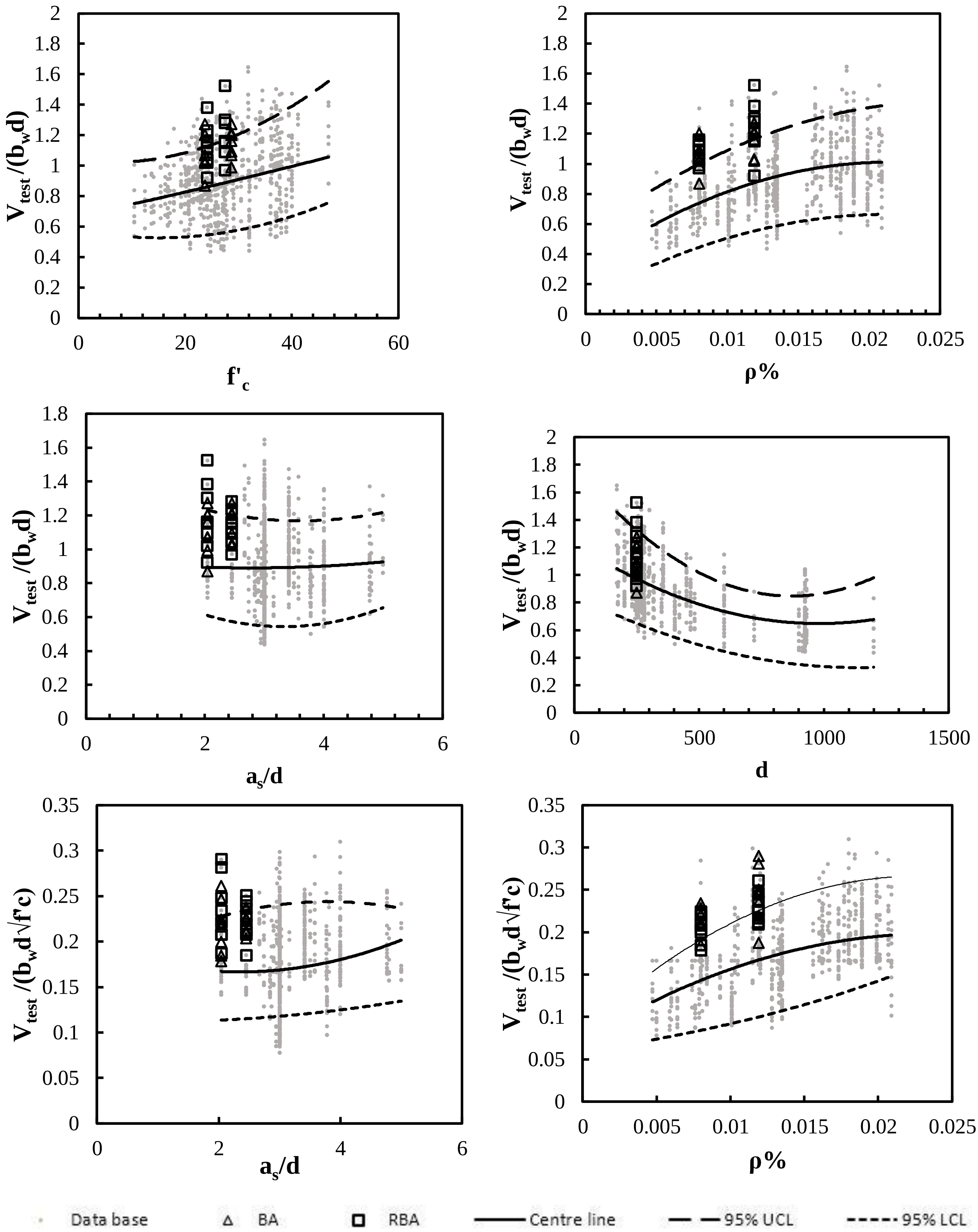


Figure 4.18: Comparison of Test Results with Shear Database

It is found that the experimental results obtained from this study are located above the average line of shear database. Some experimental data also fall above the 95% upper confidence line of shear database. Therefore, it is understood that the shear capacity of the RC beam made with BA as well as RBA are higher than the shear capacity of the beam stone aggregate concrete. Similar findings also recorded by the other authors (Akhtaruzzaman & Hasnat, 1986) though their investigation was limited to brick aggregate (BA) concrete. They also reported that the difference between the shear strength of brick aggregate concrete beams and normal weight concrete beams is more pronounced when concrete strength is low. This increase in shear strength is due to the higher tensile strength of the material. The difference is about 15 to 35% depending on the concrete strength and the span to effective depth ratio.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

This chapter includes a summary of the research findings based on discussions in Chapter 4. Moreover, recommendations and future works related to this investigation are also proposed in this chapter.

5.2 CONCLUSIONS

Based on the experimental investigations on the RC beams made with virgin brick aggregate (BA) and recycled brick aggregate (RBA), the following conclusions are drawn:

(i) No significant difference in shear capacity of the RC beams was found for beams made with RBA and BA.

(ii) Provisions of existing codes (ACI, AASHTO, CSA, BS, JSCE, Model Code and Euro code) and equations developed from the fracture mechanics approaches can be conservatively used to predict shear capacity of RC beams made with RBA and BA.

(iii) Mid-span deflection of the beams is reduced with the increase of steel ratio and compressive strength of concrete; however, it is increased with the increase of shear span to depth ratio. The same trend of results are also observed for the strain over the steel.

(iv) Irrespective of the shear-span to depth ratio, the compressive strength of concrete, effective depth, experimental results of this study fall within the range of shear database irrespective of the RBA and BA.

(v) In terms of crack morphology, crack progression, and load-deflection response, the behavior of the BA and RBA beams were virtually identical.

(vi) The AASHTO LRFD equation accurately estimate the reinforcement strain for both the BA and RBA beams.

5.3 RECOMMENDATIONS

Due to the limited number of studies of the shear behavior of RBA, further research is needed to make comparisons and conclusions across a larger database. Based on the drawn-out summaries, the following recommendations for future work can be made:

a) The scope of the research can be expanded to study the effect of creep properties and durability of concrete made with brick aggregate and recycled brick aggregate.

b) In this research limited no. of parameter variations (two variations in every parameter) has been considered. Research should be planned in a wide range of variations to understand the effect of shear strength of concrete.

c) Due to the variety of sources of RBA and the various functions, environment, and wear of the concrete structures and pavements from which the RBA can be obtained, characterizing this aggregate can be very difficult. Controlled studies must be performed to account for each of these variables on a regional basis so that the aggregates within the area can be adequately characterized.

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